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EFFECTS OF FIRST UNDERLAYER WEIGHT ON THE STABILITY OF STONE-AR--ETC(U)
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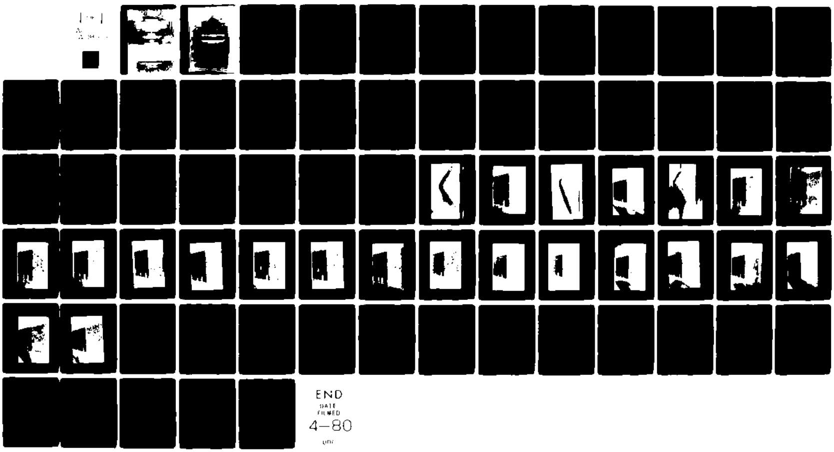

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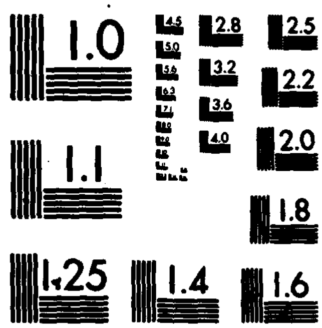

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Hydraulic model tests were conducted to determine the stability response of stone-armored breakwaters for a selected range of first underlayer weights. It was concluded that for the range of test conditions considered: (a) variations in first-underlayer stone weights (W_1) from $W_r/5$ to $W_r/20$ do not have a significant effect on armor stability; (b) armor stability will not be significantly influenced by relative depth (d/L) or wave steepness (H/L); and

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(c) wave runup and rundown are not significantly affected by variation in first-underlayer stone weights (W_1) in the range $W_r/20 \leq W_1 \leq W_r/5$.

(W sub 1)

W sub r) 20 < or = W sub 1

< or = (W sub r)/5.

PREFACE

Authority for the U. S. Army Engineer Waterways Experiment Station (WES) to conduct this study, Work Unit No. 31269, "Stability of Breakwaters," under the Corps of Engineers Civil Works Research and Development Program was contained in a letter from the Office, Chief of Engineers (OCE), U. S. Army, dated 19 May 1972. Funds were provided through the Coastal Engineering Research Area under the field managership of the Coastal Engineering Research Center and OCE Technical Monitor, Mr. J. Lockhart, HQDA (DAEN-CWE-H).

The study was conducted by personnel of the Hydraulics Laboratory, WES, under the general direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, and Dr. R. W. Whalin, Chief of the Wave Dynamics Division. Tests were conducted under the supervision of Mr. D. D. Davidson, Chief of the Wave Research Branch, by Mr. R. D. Carver, Project Engineer, and Mr. W. G. Dubose, Engineering Technician. This report was prepared by Mr. Carver.

Commanders and Directors of WES during the conduct of the study and the preparation and publication of this report were COL John L. Cannon, CE, and COL Nelson P. Conover, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) as follows:

Multiply	By	To Obtain
feet	0.3048	metres
feet per second per second	0.3048	metres per second per second
inches	25.4	millimetres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square feet	0.09290304	square metres

EFFECTS OF FIRST UNDERLAYER WEIGHT ON THE STABILITY OF STONE-ARMORED, RUBBLE-MOUND BREAKWATER TRUNKS SUBJECTED TO NONBREAKING WAVES WITH NO OVERTOPPING

Hydraulic Model Investigation

PART I: INTRODUCTION

Background

1. The hydraulic model investigation described herein constitutes a portion of a research effort to provide fundamental data for the design of rubble-mound breakwaters (both trunk and head sections) subjected to nonbreaking and breaking waves. This particular report is concerned with quarrystone armor used on breakwater trunks subjected to nonbreaking waves. In this study a rubble-mound breakwater is defined as a protective structure constructed with a core of quarry-run stone, sand, or slag and protected from wave action by one or more stone underlayers and a cover layer composed of selected quarrystone or specially shaped concrete armor units.

2. Rubble-mound breakwaters are used extensively throughout the world to provide protection from the destructive forces of storm waves for harbor and port facilities. In some locations, a proposed rubble-mound breakwater may be subject to attack by waves of such magnitude that quarrystone of adequate size to provide economic construction of a stable breakwater is not available. Under these circumstances, it is required that the protective cover layer consist of specially shaped concrete armor units.

3. In 1951, a comprehensive series of flume tests on rubble-mound breakwaters was begun at the U. S. Army Engineer Waterways Experiment Station (WES) for the Office, Chief of Engineers (OCE). This testing program included tests to provide necessary information for rubble-mound breakwater design and construction. The initial tests completed prior to September 1955 and described by Hudson (1958) dealt only with the

type of rubble-mound breakwater in which the portion of the structure subjected to the most intense wave action was protected by randomly placed quarrystone armor units.

4. Results from additional research tests conducted at WES between 1955 and 1963 were reported by Jackson (1968). These tests were concerned primarily with the stability characteristics of breakwater trunks and breakwater heads with crown elevations sufficient to prevent overtopping and with protective cover layers consisting of one or two layers of rock or specially shaped armor units. The following types of armor units were tested: smooth quarrystones (basalt), rough quarrystones (granite), tetrapods, quadripods, tribars, modified cubes, hexapods, and modified tetrahedrons.

5. In 1966, a new shape of armor unit, the dolos, was introduced (Merrifield and Zwamborn 1966) which was acclaimed to have much higher stability characteristics than any existing armor unit. Site-specific model tests of dolosse by Davidson (1971), Carver (1976), Bottin, Chatham, and Carver (1976), and Carver and Davidson (1976) showed that although the dolos stability characteristics were higher than those for existing units, they were not of the magnitude indicated by Merrifield and Zwamborn (1966) and additional data were needed to assure the design of safe and economical structures. Thus, testing of dolosse was included in a new research work unit entitled "Stability of Breakwaters" which was initiated in 1972. Comprehensive tests of dolosse, completed in 1976 and reported by Carver and Davidson (1977), showed that a stability coefficient $K = 31$ is reasonable for dolosse subjected to nonbreaking waves on breakwater trunks if the density of units per given area is equal to or greater than $0.83V^{-2/3}$, i.e., $n = 2$, $k_{\Delta} = 0.94$, and $P = 56$ percent.

6. Toskane armor, developed by Grobbelaar and tested by Retief and Vonk (1974), was claimed to have equal or higher stability characteristics than dolos armor. Therefore, based on the scope of Work Unit 31269, preliminary tests of toskane armor units were conducted (as part of Work Unit 31269) and reported by Carver (1978). These tests showed that a stability coefficient $K = 22$ is reasonable for toskane armor subjected

to nonbreaking waves on breakwater trunks if the density of units per given area is equal to or greater than $0.99\ V^{-2/3}$, i.e., $n = 2$, $k_{\Delta} = 1.03$, and $P = 56$ percent.

7. Even though several hydraulically superior armor unit shapes have been developed in recent years, quarrystone still provides the most economical armoring alternative for many structures. Much information for the design of quarrystone-armored breakwaters is available (Hudson 1958, Jackson 1968); however, these data are based on tests in which the 50 percent size of the first underlayer weight W_1 was always equal to the $W_r/10$, and this is the underlayer weight (50 percent) size recommended in the Shore Protection Manual (1977). Due to the extensive use of stone-armored structures, questions have arisen regarding the possibility of improving stability by using a larger first underlayer weight or decreasing costs by using a smaller first underlayer weight.

Purpose of Study

8. The purpose of the present investigation was to determine the stability response of stone-armored breakwaters for a selected range of first underlayer weights. More specifically, it was desired to quantify, as a function of first underlayer weight, variations in the following parameters:

- a. The stability coefficient K.
- b. Wave runup R_u.
- c. Wave rundown R_d.

PART II: *ANALYTICAL CONSIDERATIONS*

Stability of Rubble-Mound Breakwaters

9. When short-period waves attack rubble-mound breakwaters the interaction of the dislodging forces induced by the water motion and the resistive action of the armor units creates a complex dynamic phenomenon. Previous attempts to theoretically analyze this phenomenon to ascertain the magnitude of the dynamic forces involved have not been successful; however, hydraulic scale models of breakwaters can yield accurate design information that relates the required weight of individual breakwater armor units to breakwater geometry, local bathymetry, wave characteristics, etc.

10. The principal force tending to dislodge armor units from the breakwater slope under short-period wave attack is the drag force (F_d) while the principal resistive force is the buoyant weight $\left(W_r^1\right)$ of individual armor units and at the instant of incipient instability $F_d = W_r^1$. *Hudson (1958) has shown that equating the appropriate forms of the drag* force and buoyant weight equations develops the following functional relation

$$\frac{\gamma_r^{1/3} H}{(S_r - 1)(W_r)^{1/3}} = f\left(\cot \alpha \text{ , } \Delta \text{ , } \frac{d}{L} \text{ , } \frac{H}{L} \text{ , } D \text{ , } PT \text{ , } R_N \text{ , } P\right) \qquad (1)$$

where

γ_r = unit weight of an armor unit

H = wave height

$S_r = \gamma_r/\gamma_w$ is the specific gravity of an armor unit relative to the water in which the breakwater is constructed

W_r = weight of an armor unit in air

$\cot \alpha$ = reciprocal of breakwater slope

Δ = shape factor of the armor unit

d/L = relative depth

H/L = wave steepness

D = damage parameter

PT = technique used to place armor units in the cover layer

R_N = Reynolds stability number = $\left(g^{1/2}H^{1/2}\ell_a\right)/\nu$

g = acceleration due to gravity

ℓ_a = characteristic length = $k_\Delta(W/\gamma)^{1/3}$

ν = kinematic viscosity

P = porosity of the armor layer and underlayers

For the present investigation, porosity can be conveniently represented by the relative underlayer weight (W_r/W_1). Therefore, correlation of the stability test results will be attempted by the following functional relation

$$\frac{\gamma_r^{1/3}H}{(S_r - 1)(W_r)^{1/3}} = f\left(\cot\alpha , \Delta , \frac{d}{L} , \frac{H}{L} , D , PT , R_N , \frac{W_r}{W_1}\right) \tag{2}$$

Wave Runup and Rundown

11. Before a breakwater design can be optimized, it is necessary for the designer to be able to accurately estimate R_u and R_d for the anticipated range of wave conditions to which the structure will be subjected. Runup data are useful in selecting a crown elevation that will prevent excessive wave overtopping, and rundown data are useful in selecting the minimum depth below the still-water level (swl) to which the armor units should extend to prevent failure of the cover layer.

12. The primary variables affecting wave runup on sloping structures are $\cot\alpha$, H , d , L , and P , i.e.

$$R_u = f(\cot\alpha , H , d , L , P) \tag{3}$$

One possible set of pi terms is

$$\pi_1 = \frac{Ru}{H} \tag{4}$$

$$\pi_2 = \frac{H}{L} \tag{5}$$

$$\pi_3 = \frac{d}{L} \tag{6}$$

$$\pi_4 = \cot \alpha \tag{7}$$

$$\pi_5 = P \tag{8}$$

Correlation of the test data will be attempted by the functional relation

$$\pi_1 = f(\pi_2 , \pi_3 , \pi_4 , \pi_5) \tag{9}$$

or

$$\frac{R_u}{H} = f\left(\frac{H}{L} , \frac{d}{L} , \cot \alpha , P\right) \tag{10}$$

and finally representing P as W_r/W_1

$$\frac{R_u}{H} = f\left(\frac{H}{L} , \frac{d}{L} , \cot \alpha , \frac{W_r}{W_1}\right) \tag{11}$$

Assuming that the primary variables affecting wave rundown are the same as those affecting wave runup, a similar analysis will yield the following functional relation

$$\frac{R_d}{H} = f\left(\frac{H}{L} , \frac{d}{L} , \cot \alpha , \frac{W_r}{W_1}\right) \tag{12}$$

Stability Scale Effects

13. If the absolute sizes of breakwater materials and wave dimensions become too small, flow around the armor units enters the laminar regime and the induced drag forces become a direct function of the Reynolds number. Under these circumstances prototype phenomena are not properly simulated and stability scale effects are induced. A detailed discussion of the design requirements necessary to ensure the preclusion of stability scale effects in small-scale breakwater models is presented

by Hudson (1975) (critical $R_N = 3 \times 10^4$). For all tests reported herein the sizes of model armor and wave dimensions were selected such that scale effects were insignificant (i.e., R_N was greater than 3×10^4).

PART III: TESTS

Tests Conducted

14. Tests were conducted to determine the effects of first underlayer weight on the stability of stone armor used on breakwater trunks and subjected to nonbreaking waves. In these tests, the maximum wave heights that caused no more than 5 percent damage to the cover layers were determined on breakwater sections with crown elevations high enough to prevent overtopping by the test waves. Sections of the type shown in Plates 1 and 2 and Photos 1-4 were used for all tests. Tests also were conducted to determine the extent of wave runup and rundown on the structures investigated.

Test Procedures

Methods of constructing test sections

15. All model breakwater sections were constructed to reproduce as closely as possible the results of the usual methods of constructing prototype breakwaters. The core material was dampened as it was dumped by bucket or shovel into the flume and was compacted with hand trowels to simulate natural consolidation resulting from wave action during construction of the prototype structure. Once the core material was in place, it was sprayed with a low-velocity water hose to ensure adequate compaction of the material. The underlayer stone was then added by shovel and smoothed to grade by hand or with trowels. No excessive pressure or compaction was applied during placement of the underlayer stone. Armor units used in the cover layers were placed in a random manner, i.e., laid down in such a way that no intentional interlocking of the units was obtained. After each test, the armor stones were removed from the breakwater, all of the underlayer stones were replaced to the grade of the original test section, and the armor stones were replaced.

Method of determining damage

16. In order to evaluate and compare breakwater stability test

results, it is necessary to quantify the changes that have taken place in a given structure during attack by waves of specified characteristics. During the early 1950's, WES developed a method of measuring the percent damage incurred by a test section. This method has proven satisfactory and is used as a means for analyzing and comparing the stability tests delineated herein.

17. The WES damage-measurement technique requires that the cross-sectional area occupied by armor units be determined for each stability test section. Armor unit area is computed from elevations (soundings) taken at preselected locations over the seaward face of the structure before the armor is placed on the underlayer, after the armor has been placed but before the section has been subjected to wave attack, and finally after wave attack. Elevations are obtained with a sounding rod equipped with a circular spirit level for plumbing, a scale graduated in thousandths of a foot, and a ball-and-socket foot for adjustment to the irregular surface of the breakwater slope. The diameter (diam) in inches of the circular foot of the sounding rod was related to the size of the material being sounded by the following equation:

$$\text{diam} = 6.8\left(\frac{W_r}{\gamma_r}\right)^{1/3} \qquad (13)$$

A series of sounding tests in which both the weight of the stone and the diameter of the sounding foot were varied indicated that the above relation would give a measured thickness which visually appeared to represent an acceptable two-layer thickness.

18. Sounding data for each test section were obtained as follows: after the first underlayer was in place, soundings were taken on the sea-side slope of the structure along rows beginning at and parallel to the longitudinal center line of the structure and extending in 0.25-ft* horizontal increments until a line was reached that approximated the

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* A table of factors for converting U. S. customary units of measurements to metric (SI) units is presented on page 3.

location of maximum wave rundown. On each parallel row, 13 sounding points spaced at 0.25-ft increments were measured. This distance represented the middle 3 ft of a 5-ft-wide test section; the 1 ft of structure next to each wall was not considered because of the possibility of discontinuity effects between the armor units and the flume walls. Soundings were taken at the same points once the armor was in place and again after the structure had been subjected to wave attack.

19. Sounding data from each stability test were reduced in the following manner. The individual sounding points obtained on each parallel row were averaged to yield an average elevation at the bottom of the armor layer before the dolosse were placed and then at the top of the armor layer before and after testing. From these values, the cross-sectional armor area before testing and the area from which armor units were displaced (either downslope or off the section) were calculated. Damage was then determined from the following relation:

$$\text{Percent damage} = \frac{A_2}{A_1}(100) \qquad (14)$$

where

A_1 = area before testing, ft^2

A_2 = area from which armor units have been displaced, ft^2

The percentage given by the WES sounding technique is, therefore, a measurement of an end area which converts to an average volume of armor material that has been moved from its original location (either downslope or off the structure). This particular method of measuring damage does not consider the rocking of individual armor units as exercised by some researchers. However, WES visual definition of no-damage from which the less than 5 percent displaced volume criterion determined by the sounding technique was developed is defined such that no significant movement of individual units is allowed; thus the rocking criterion does not play as important a part in our evaluation as those of some other researchers.

<u>Selection of design wave heights</u>

20. Design wave heights for the no-damage criterion were determined by subjecting the test sections to monochromatic waves,

successively larger in height in 0.01- to 0.02-ft increments, until the maximum wave height was found that would produce no more than 5 percent damage. Each test wave was allowed to attack the breakwater for a cumulative period of 30 min, then the test sections were rebuilt prior to attack by the next added increment wave. This 30-min interval allowed sufficient time for the test sections to stabilize, i.e., time for all significant movement of armor material to abate. During the tests, the wave generator was stopped as soon as reflected waves from the breakwater reached it, and the waves were allowed to decay to zero height before restarting the generator in order to prevent the test section from being exposed to an undefined set of wave conditions.

Measurement of
wave runup and rundown

21. Values of R_u and R_d were obtained with a point gage calibrated in increments of 0.001 ft and mounted on an aluminum framework which could be moved along and across the seaward breakwater slope. Due to slight height variations from wave to wave within a given wave train and the highly porous texture of the breakwater slope, at least three measurements of R_u and R_d were made for each test wave condition. These measurements were later averaged to yield R_u and R_d values for each of the selected wave conditions. Photo 5 shows the runup produced by a 2.65-sec, 0.50-ft wave on a 1:1.5 slope with $W_1 = W_r/5$.

Test Equipment and Materials

Equipment used

22. All wave action tests were conducted in a flat-bottomed, 5-ft-wide, 4-ft-deep, and 119-ft-long concrete wave flume with test sections installed in the flume about 90 ft from a vertical displacement wave generator. The generator is capable of producing sinusoidal waves of various periods and heights. Test waves of the required characteristics were generated by varying the frequency and amplitude of the plunger motion. Changes in water-surface elevation as a function of time were measured by electrical wave-height gages in the vicinity where the toe

of the test sections was to be placed and recorded on chart paper by an electrically operated oscillograph. The electrical output of the wave gages was directly proportional to their submergence depth.

<u>Materials used</u>

23. Rough granite stone (W_r) with an average length of approximately two times its width, an average weight of 0.55 lb ($\pm$0.025 lb), and a specific weight of 167.0 pcf was used to armor the model breakwater sections. Sieve-sized limestone (γ = 165.0 pcf) of angular shape was used for the underlayers (W_1 and W_2) and the core (W_3).

PART IV: TEST RESULTS

Stability Tests

24. Results of stability tests using nonbreaking waves, and for the no-damage and no-overtopping criteria, are summarized in Table 1. This table contains the experimentally determined design wave heights and corresponding stability numbers as functions of relative underlayer weight, breakwater slope, relative depth, and wave steepness. All stability test results presented in Table 1 were verified by one repeat test. In these tests, the sea-side breakwater slopes were 1:1.5 and 1:3; relative depths ranged from 0.10 to 0.25; W_1 values were $W_r/5$, $W_r/10$, and $W_r/20$; wave heights ranged from 0.50 to 0.63 ft; water depth was 2.0 ft; and the number of armor units per given surface area, A, was $N = 1.45V^{-2/3}$ ($k_\Delta = 1.15$ and $P = 37$ percent). Photos 6-23 show the after-testing stability condition of the structures.

25. As discussed in paragraph 10, it was hoped that stability test results could be analyzed by the following functional relation for the stability number, N_S, where

$$N_S = \frac{\gamma_r^{1/3} H}{(S_r - 1) W_r^{1/3}} = f\left(\cot\alpha , \Delta , \frac{d}{L} , \frac{H}{L} , D , PT , R_N , \frac{W_r}{W_1}\right) \tag{15}$$

For tests described herein Δ, D, and PT were held constant; therefore, Equation 15 reduces to

$$N_S = f\left(\cot\alpha , \frac{d}{L} , \frac{H}{L} , R_N , \frac{W_r}{W_1}\right) \tag{16}$$

Also, as described in paragraph 13, the sizes of model armor units and wave dimensions were selected such that turbulent flow was always obtained; therefore, N_S was independent of R_N and Equation 16 becomes

$$N_S = f\left(\cot\alpha , \frac{d}{L} , \frac{H}{L} , \frac{W_r}{W_1}\right) \tag{17}$$

26. Plots of N_S versus d/L and H/L are presented in Plates 3 and 4, respectively. These data show N_S to be independent of both d/L and H/L , for constant values of W_r/W_1 and cot α . Plate 5 presents plots of N_S versus W_r/W_1 for constant values of cot α . For the range of first underlayer weights investigated, these data show N_S to be independent of W_r/W_1 . Based on the data presented in Plates 3-5, Equation 17 reduces to

$$N_S = f(\cot \alpha) \tag{18}$$

27. Plate 6 presents a log-log plot of N_S versus cot α and the lines AB and CD are average and lower limit fits to the data points using a slope of 1/3. The general equation of a line on log-log paper is of the form

$$y = ax^b \tag{19}$$

where a is the y intercept at $x = 1$, and b is the slope of the line. Line AB has a y intercept of 1.77 and a slope of 1/3; therefore, the equation of line AB must be

$$N_S = 1.77 \ (\cot \alpha)^{1/3} \tag{20}$$

or

$$\frac{N_S^3}{\cot \alpha} = 5.5 \tag{21}$$

Substituting

$$N_S = \frac{\gamma_r^{1/3} H}{(S_r - 1) W_r^{1/3}}$$

and rearranging, Equation 21 becomes

$$W_r = \frac{\gamma_r H^3}{5.5(S_r - 1)^3 \cot \alpha} \tag{22}$$

Equation 22 is immediately recognized as the Hudson Stability Equation (Hudson 1958) with $K = 5.5$. Line CD has a y intercept of 1.72 and a slope of 1/3; consequently, the equation of line CD is

$$N_S = 1.72 (\cot \alpha)^{1/3} \tag{23}$$

or

$$\frac{N_S^3}{\cot \alpha} = 5.1 \tag{24}$$

Again substituting and rearranging, Equation 24 becomes

$$W_r = \frac{\gamma_r H^3}{5.1(S_r - 1)^3 \cot \alpha} \tag{25}$$

The data analysis presented herein shows good correlation of stability test results by the Hudson Stability Equation with average and lower limit stability coefficients (K values) of 5.5 and 5.1, respectively.

Wave Runup and Rundown Tests

28. Runup, average runup, and the standard deviation are shown in Table 2 for all test conditions. Rundown data are treated in a similar manner in Table 3. Considering the small random variation inherent in test waves within a given wave train and small local variation in the texture and porosity of the breakwater slope, the test results appear to be quite consistent.

29. As described in paragraph 12, it was hoped that runup and rundown test results could be correlated by functional relations for relative runup (R_u/H) and relative rundown (R_d/H), i.e.

$$\frac{R_u}{H} = f\left(\frac{H}{L}, \frac{d}{L}, \cot \alpha, \frac{W_r}{W_1}\right) \qquad (11\ \text{bis})$$

and

$$\frac{R_d}{H} = f\left(\frac{H}{L}, \frac{d}{L}, \cot \alpha, \frac{W_r}{W_1}\right) \qquad (12\ \text{bis})$$

Calculated values of relative runup and relative rundown along with corresponding values of relative depth and wave steepness are presented in Table 4 using the average runup and rundown from Tables 2 and 3. Plates 7 and 8 present R_u/H as a function of H/L while Plates 9 and 10 present R_u/H as a function of d/L. Plots of R_d/H versus H/L are given in Plates 11 and 12 and Plates 13 and 14 present plots of R_d/H versus d/L. These data show neither R_u nor R_d to be significantly influenced by W_r/W_1 for the range of underlayer weights investigated. However, they do show both R_u and R_d to be functions of breakwater slope, wave steepness, and relative depth. Vanoni and Raichlen (1966) have shown that for relative wave heights (H/d) from about 0.05 to 0.5 on breakwater sections of stone and tribars, relative runup increased to some extent with H/d. However, in the runup tests described herein, for which H/d ranged from 0.1 to 0.3, the effects of H/d on R_u/H were not apparent.

30. Hudson (1958) found that when relative runup for nonbreaking waves is plotted against H/L, the shape of the curve is concave; i.e., for small values of H/L of about 0.01, R_u/H is relatively large and as H/L increases R_u/H increases to a maximum value and then decreases as H/L continues to increase. Tests conducted by Jackson (1968) indicate that R_u/H and R_d/H generally decrease with increasing H/L, with the trend being considerably more apparent with R_d/H. Runup and rundown data for dolos armor, obtained by Carver and Davidson (1977), showed the same trends as those presented by Jackson (1968). Also, tests conducted by Bottin, Chatham, and Carver (1976) on dolos armor for relatively long-period waves (d/L less than 0.10) showed the same trends as those presented by Jackson (1968) and Carver and Davidson (1977).

31. The data presented in Table 4 and Plates 7-14 show several distinct trends similar to those presented by Hudson (1958), Jackson (1968), Bottin, Chatham, and Carver (1976), and Carver and Davidson (1977). Plots of R_u/H versus H/L, given in Plates 7 and 8, show trends similar to those noted by Hudson (1958). The general trend for both R_u/H and R_d/H to decrease with increasing values of H/L are in agreement with Jackson (1968), Bottin, Chatham, and Carver (1976), and Carver and Davidson (1977). Also, the trends for both R_u/H and R_d/H to decrease with increasing values of d/L are consistent with Carver and Davidson (1977).

32. Data presented herein show both R_u/H and R_d/H to be dependent upon H/L and d/L; however, it appears that R_d/H is most affected by H/L and R_u/H is most affected by d/L. Flattening the slope from 1:1.5 to 1:3 generally reduced both R_u and R_d. The general tendency for both runup and rundown to decrease at the milder slope seems reasonable since as the slope becomes flatter the wave has a longer travel distance to reach a given elevation and, therefore, a greater opportunity to dissipate energy.

PART V: CONCLUSIONS

33. Based on the tests and results described herein, in which stone armor is used on breakwater trunks and subjected to nonbreaking waves with a direction of approach of 90°, it is concluded that:

- a. Variations in first-underlayer stone weights (W_1) from $W_r/5$ to $W_r/20$ do not have a significant effect on armor stability.
- b. Armor stability will not be significantly influenced by relative depth (d/L) or wave steepness (H/L) over the range of conditions tested ($0.10 \leq d/L \leq 0.25$ and $0.026 \leq H/L \leq 0.079$).
- c. Stability test results are well correlated by the Hudson Stability Equation, i.e., $N_S = (K \cot \alpha)^{1/3}$.
- d. Wave runup and rundown are not significantly affected by variations in first-underlayer stone weights (W_1) in the range $W_r/20 \leq W_1 \leq W_r/5$.
- e. Wave relative runup (R_u/H) and relative rundown (R_d/H) are functions of wave steepness (H/L), relative depth (d/L), and breakwater slope.

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Table 1

Values of $H_{D=0}$ and N_S for Two Layers of Stone Armor Randomly Placed on Breakwater Trunks and Subjected to Nonbreaking Waves with No Overtopping: W_r = 0.55 lb; γ_r = 167 pcf; d = 2.0 ft; $W_1 = W_r/5$, $W_r/10$, and $W_r/20$

d/L	H/L	T sec	$H_{D=0}$ ft	N_S	Percent Damage Test 1	Percent Damage Test 2	Percent Damage Average
			cot α = 1.5 ; $W_1 = W_r/5$				
0.10	0.026	2.65	0.51	2.04	1.1	2.1	1.6
0.15	0.038	1.89	0.51	2.04	1.9	0.4	1.2
0.25	0.064	1.31	0.51	2.04	1.9	1.5	1.7
			cot α = 1.5 ; $W_1 = W_r/10$				
0.10	0.026	2.65	0.51	2.04	1.9	0.9	1.4
0.15	0.039	1.89	0.52	2.09	1.5	1.3	1.4
0.25	0.065	1.31	0.52	2.09	1.0	1.4	1.2
			cot α = 1.5 ; $W_1 = W_r/20$				
0.10	0.026	2.65	0.51	2.04	0.9	1.6	1.3
0.15	0.039	1.89	0.52	2.09	1.3	0.9	1.1
0.25	0.063	1.31	0.50	2.00	1.2	2.0	1.6
			cot α = 3.0 ; $W_1 = W_r/5$				
0.15	0.047	1.89	0.63	2.53	1.3	0.9	1.1
0.20	0.063	1.52	0.63	2.53	1.1	1.5	1.3
0.25	0.079	1.31	0.63	2.53	0.7	1.7	1.2
			cot α = 3.0 ; $W_1 = W_r/10$				
0.15	0.047	1.89	0.63	2.53	2.8	0.8	1.8
0.20	0.062	1.52	0.62	2.49	1.5	1.7	1.6
0.25	0.079	1.31	0.63	2.53	1.1	2.1	1.6
			cot α = 3.0 ; $W_1 = W_r/20$				
0.15	0.047	1.89	0.62	2.49	1.3	1.5	1.4
0.20	0.062	1.52	0.62	2.49	0.9	1.6	1.3
0.25	0.078	1.31	0.62	2.49	1.4	1.0	1.2

Table 2

Wave Runup (R_u) Data for Quarrystone Armor Randomly Placed on Breakwater

Trunks and Subjected to Nonbreaking Waves with No Overtopping:

Cot α = 1.5 and 3.0 ; $W_1 = W_r/5$, $W_r/10$, and $W_r/20$

d/L	T sec	H ft	R_u, ft Test 1	Test 2	Test 3	Average	Standard Deviation ft
			Cot α = 1.5 ; $W_1 = W_r/5$				
0.10	2.65	0.20	0.23	0.22	0.22	0.22	0.007
0.10	2.65	0.35	0.39	0.41	0.40	0.40	0.010
0.10	2.65	0.50	0.59	0.61	0.58	0.59	0.016
0.25	1.31	0.25	0.26	0.26	0.27	0.26	0.007
0.25	1.31	0.35	0.35	0.37	0.36	0.36	0.010
0.25	1.31	0.45	0.39	0.44	0.44	0.42	0.029
0.40	0.99	0.30	0.24	0.23	0.23	0.23	0.007
0.40	0.99	0.40	0.32	0.25	0.28	0.28	0.035
			Cot α = 1.5 ; $W_1 = W_r/10$				
0.10	2.65	0.20	0.22	0.23	0.23	0.23	0.007
0.10	2.65	0.35	0.42	0.43	0.40	0.42	0.016
0.10	2.65	0.50	0.62	0.62	0.58	0.61	0.023
0.25	1.31	0.25	0.25	0.23	0.24	0.24	0.010
0.25	1.31	0.35	0.34	0.34	0.35	0.34	0.007
0.25	1.31	0.45	0.43	0.44	0.42	0.43	0.010
0.40	0.99	0.30	0.22	0.21	0.22	0.22	0.007
0.40	0.99	0.40	0.27	0.28	0.26	0.27	0.010
			Cot α = 1.5 ; $W_1 = W_r/20$				
0.10	2.65	0.20	0.22	0.23	0.23	0.23	0.007
0.10	2.65	0.35	0.42	0.42	0.43	0.42	0.007
0.10	2.65	0.50	0.59	0.62	0.61	0.61	0.016
0.25	1.31	0.25	0.23	0.22	0.24	0.23	0.010
0.25	1.31	0.35	0.36	0.34	0.39	0.36	0.025
0.25	1.31	0.45	0.44	0.46	0.42	0.44	0.020
0.40	0.99	0.30	0.17	0.19	0.19	0.18	0.012
0.40	0.99	0.40	0.30	0.30	0.29	0.30	0.007

(Continued)

Table 2 (Concluded)

d/L	T sec	H ft	R_u, ft Test 1	Test 2	Test 3	Average	Standard Deviation ft
			Cot α = 3.0 ; $W_1 = W_r/5$				
0.10	2.65	0.20	0.25	0.25	0.25	0.25	0.000
0.10	2.65	0.35	0.42	0.43	0.43	0.43	0.007
0.10	2.65	0.50	0.58	0.64	0.58	0.60	0.035
0.10	2.65	0.60	0.74	0.70	0.72	0.72	0.020
0.25	1.31	0.25	0.22	0.21	0.22	0.22	0.007
0.25	1.31	0.35	0.28	0.29	0.27	0.28	0.010
0.25	1.31	0.45	0.36	0.36	0.36	0.36	0.000
0.25	1.31	0.55	0.41	0.40	0.41	0.41	0.007
0.40	0.99	0.30	0.18	0.18	0.18	0.18	0.000
0.40	0.99	0.40	0.26	0.28	0.21	0.25	0.036
0.40	0.99	0.45	0.27	0.26	0.28	0.27	0.010
			Cot α = 3.0 ; $W_1 = W_r/10$				
0.10	2.65	0.20	0.23	0.25	0.24	0.24	0.010
0.10	2.65	0.35	0.44	0.40	0.47	0.44	0.035
0.10	2.65	0.50	0.63	0.56	0.61	0.60	0.036
0.10	2.65	0.60	0.71	0.70	0.70	0.70	0.007
0.25	1.31	0.25	0.20	0.21	0.20	0.20	0.007
0.25	1.31	0.35	0.31	0.28	0.27	0.29	0.021
0.25	1.31	0.45	0.37	0.35	0.37	0.36	0.012
0.25	1.31	0.55	0.42	0.43	0.40	0.42	0.016
0.40	0.99	0.30	0.18	0.19	0.19	0.19	0.007
0.40	0.99	0.40	0.23	0.20	0.22	0.22	0.016
0.40	0.99	0.45	0.30	0.26	0.27	0.28	0.021
			Cot α = 3.0 ; $W_1 = W_r/20$				
0.10	2.65	0.20	0.26	0.23	0.22	0.24	0.021
0.10	2.65	0.35	0.43	0.41	0.43	0.42	0.012
0.10	2.65	0.50	0.60	0.59	0.59	0.59	0.007
0.10	2.65	0.60	0.67	0.68	0.66	0.67	0.010
0.25	1.31	0.25	0.19	0.18	0.20	0.19	0.010
0.25	1.31	0.35	0.28	0.26	0.26	0.27	0.012
0.25	1.31	0.45	0.33	0.33	0.34	0.33	0.007
0.25	1.31	0.55	0.36	0.38	0.38	0.37	0.012
0.40	0.99	0.30	0.17	0.16	0.15	0.16	0.010
0.40	0.99	0.40	0.24	0.21	0.20	0.22	0.021
0.40	0.99	0.45	0.25	0.22	0.21	0.23	0.021

Table 3

Wave Rundown (R_d) Data for Quarrystone Armor Randomly Placed on Breakwater Trunks and Subjected to Nonbreaking Waves with No Overtopping: Cot α = 1.5 and 3.0 ; $W_1 = W_r/5$, $W_r/10$, and $W_r/20$

d/L	T sec	H ft	R_d, ft Test 1	Test 2	Test 3	Average	Standard Deviation ft
			Cot α = 1.5 ; $W_1 = W_r/5$				
0.10	2.65	0.20	0.22	0.20	0.22	0.21	0.012
0.10	2.65	0.35	0.35	0.36	0.35	0.35	0.007
0.10	2.65	0.50	0.47	0.42	0.44	0.44	0.025
0.25	1.31	0.25	0.21	0.17	0.19	0.19	0.020
0.25	1.31	0.35	0.27	0.26	0.29	0.27	0.016
0.25	1.31	0.45	0.32	0.32	0.31	0.32	0.007
0.40	0.99	0.30	0.15	0.14	0.16	0.15	0.010
0.40	0.99	0.40	0.22	0.19	0.22	0.21	0.017
			Cot α = 1.5 ; $W_1 = W_r/10$				
0.10	2.65	0.20	0.21	0.22	0.20	0.21	0.010
0.10	2.65	0.35	0.33	0.33	0.31	0.32	0.012
0.10	2.65	0.50	0.42	0.42	0.40	0.41	0.012
0.25	1.31	0.25	0.20	0.23	0.21	0.21	0.016
0.25	1.31	0.35	0.21	0.28	0.22	0.24	0.038
0.25	1.31	0.45	0.27	0.29	0.25	0.27	0.020
0.40	0.99	0.30	0.14	0.16	0.14	0.15	0.012
0.40	0.99	0.40	0.17	0.19	0.19	0.18	0.012
			Cot α = 1.5 ; $W_1 = W_r/20$				
0.10	2.65	0.20	0.21	0.21	0.21	0.21	0.000
0.10	2.65	0.35	0.36	0.34	0.34	0.35	0.012
0.10	2.65	0.50	0.48	0.43	0.47	0.46	0.026
0.25	1.31	0.25	0.21	0.21	0.20	0.21	0.007
0.25	1.31	0.35	0.25	0.23	0.24	0.24	0.010
0.25	1.31	0.45	0.28	0.27	0.27	0.27	0.007
0.40	0.99	0.30	0.15	0.17	0.17	0.16	0.012
0.40	0.99	0.40	0.23	0.17	0.21	0.20	0.031

(Continued)

Table 3 (Concluded)

d/L	T sec	H ft	R_d, ft Test 1	Test 2	Test 3	Average	Standard Deviation ft
			Cot α = 3.0 ; $W_1 = W_r/5$				
0.10	2.65	0.20	0.19	0.19	0.20	0.19	0.007
0.10	2.65	0.35	0.29	0.28	0.31	0.29	0.016
0.10	2.65	0.50	0.33	0.32	0.34	0.33	0.010
0.10	2.65	0.60	0.40	0.36	0.40	0.39	0.023
0.25	1.31	0.25	0.12	0.15	0.14	0.14	0.016
0.25	1.31	0.35	0.15	0.16	0.14	0.15	0.010
0.25	1.31	0.45	0.17	0.14	0.15	0.15	0.016
0.25	1.31	0.55	0.17	0.17	0.15	0.16	0.012
0.40	0.99	0.30	0.11	0.08	0.11	0.10	0.017
0.40	0.99	0.40	0.13	0.14	0.13	0.13	0.007
0.40	0.99	0.45	0.14	0.12	0.13	0.13	0.010
			Cot α = 3.0 ; $W_1 = W_r/10$				
0.10	2.65	0.20	0.19	0.22	0.19	0.20	0.017
0.10	2.65	0.35	0.26	0.25	0.27	0.26	0.010
0.10	2.65	0.50	0.34	0.34	0.32	0.33	0.012
0.10	2.65	0.60	0.36	0.39	0.35	0.37	0.021
0.25	1.31	0.25	0.14	0.14	0.15	0.14	0.007
0.25	1.31	0.35	0.15	0.16	0.14	0.15	0.010
0.25	1.31	0.45	0.16	0.15	0.18	0.16	0.016
0.25	1.31	0.55	0.15	0.19	0.16	0.17	0.021
0.40	0.99	0.30	0.09	0.11	0.11	0.10	0.012
0.40	0.99	0.40	0.12	0.09	0.14	0.12	0.025
0.40	0.99	0.45	0.14	0.11	0.15	0.13	0.021
			Cot α = 3.0 ; $W_1 = W_r/20$				
0.10	2.65	0.20	0.21	0.18	0.18	0.19	0.017
0.10	2.65	0.35	0.24	0.28	0.26	0.26	0.020
0.10	2.65	0.50	0.33	0.34	0.29	0.32	0.026
0.10	2.65	0.60	0.38	0.34	0.37	0.36	0.021
0.25	1.31	0.25	0.11	0.14	0.11	0.12	0.017
0.25	1.31	0.35	0.14	0.12	0.14	0.13	0.012
0.25	1.31	0.45	0.14	0.15	0.13	0.14	0.010
0.25	1.31	0.55	0.19	0.11	0.15	0.15	0.040
0.40	0.99	0.30	0.08	0.11	0.08	0.09	0.017
0.40	0.99	0.40	0.09	0.09	0.13	0.10	0.023
0.40	0.99	0.45	0.13	0.13	0.11	0.12	0.012

Table 4

Comparative Values of R_u/H and R_d/H for Quarrystone Armor Randomly Placed on Breakwater Trunks and Subjected to Nonbreaking Waves with No Overtopping: Cot α = 1.5 and 3.0 ; $W_1 = W_r/5$, $W_r/10$, and $W_r/20$

d/L	T sec	H ft	H/L	R_u* ft	R_u/H	R_d* ft	R_d/H
			Cot α = 1.5 ; $W_1 = W_r/5$				
0.10	2.65	0.20	0.010	0.22	1.10	0.21	1.05
0.10	2.65	0.35	0.018	0.40	1.14	0.35	1.00
0.10	2.65	0.50	0.025	0.59	1.18	0.44	0.88
0.25	1.31	0.25	0.031	0.26	1.04	0.19	0.76
0.25	1.31	0.35	0.044	0.36	1.03	0.27	0.77
0.25	1.31	0.45	0.056	0.42	0.93	0.32	0.71
0.40	0.99	0.30	0.060	0.23	0.77	0.15	0.50
0.40	0.99	0.40	0.080	0.28	0.70	0.21	0.53
			Cot α = 1.5 ; $W_1 = W_r/10$				
0.10	2.65	0.20	0.010	0.23	1.15	0.21	1.05
0.10	2.65	0.35	0.018	0.42	1.20	0.32	0.91
0.10	2.65	0.50	0.025	0.61	1.22	0.41	0.82
0.25	1.31	0.25	0.031	0.24	0.96	0.21	0.84
0.25	1.31	0.35	0.044	0.34	0.97	0.24	0.69
0.25	1.31	0.45	0.056	0.43	0.96	0.27	0.60
0.40	0.99	0.30	0.060	0.22	0.73	0.15	0.50
0.40	0.99	0.40	0.080	0.27	0.68	0.18	0.45
			Cot α = 1.5 ; $W_1 = W_r/20$				
0.10	2.65	0.20	0.010	0.23	1.15	0.21	1.05
0.10	2.65	0.35	0.018	0.42	1.20	0.35	1.00
0.10	2.65	0.50	0.025	0.61	1.22	0.46	0.92
0.25	1.31	0.25	0.031	0.23	0.92	0.21	0.84
0.25	1.31	0.35	0.044	0.36	1.03	0.24	0.69
0.25	1.31	0.45	0.056	0.44	0.98	0.27	0.60
0.40	0.99	0.30	0.060	0.18	0.60	0.16	0.53
0.40	0.99	0.40	0.080	0.30	0.75	0.20	0.50

(Continued)

* R_u and R_d represent the average values from three tests shown in Tables 2 and 3.

Table 4 (Concluded)

d/L	T sec	H ft	H/L	R_u^* ft	R_u/H	R_d^* ft	R_d/H
			Cot α = 3.0 ; $W_1 = W_r/5$				
0.10	2.65	0.20	0.010	0.25	1.25	0.19	0.95
0.10	2.65	0.35	0.018	0.43	1.23	0.29	0.83
0.10	2.65	0.50	0.025	0.60	1.20	0.33	0.66
0.10	2.65	0.60	0.030	0.72	1.20	0.39	0.65
0.25	1.31	0.25	0.031	0.22	0.88	0.14	0.56
0.25	1.31	0.35	0.044	0.28	0.80	0.15	0.43
0.25	1.31	0.45	0.056	0.36	0.80	0.15	0.33
0.25	1.31	0.55	0.069	0.41	0.75	0.16	0.29
0.40	0.99	0.30	0.060	0.18	0.60	0.10	0.33
0.40	0.99	0.40	0.080	0.25	0.63	0.13	0.33
0.40	0.99	0.45	0.090	0.27	0.60	0.13	0.29
			Cot α = 3.0 ; $W_1 = W_r/10$				
0.10	2.65	0.20	0.010	0.24	1.20	0.20	1.00
0.10	2.65	0.35	0.018	0.44	1.26	0.26	0.74
0.10	2.65	0.50	0.025	0.60	1.20	0.33	0.66
0.10	2.65	0.60	0.030	0.70	1.17	0.37	0.62
0.25	1.31	0.25	0.031	0.20	0.80	0.14	0.56
0.25	1.31	0.35	0.044	0.29	0.83	0.15	0.43
0.25	1.31	0.45	0.056	0.36	0.80	0.16	0.36
0.25	1.31	0.55	0.069	0.42	0.76	0.17	0.31
0.40	0.99	0.30	0.060	0.19	0.63	0.10	0.33
0.40	0.99	0.40	0.080	0.22	0.55	0.12	0.30
0.40	0.99	0.45	0.090	0.28	0.62	0.13	0.29
			Cot α = 3.0 ; $W_1 = W_r/20$				
0.10	2.65	0.20	0.010	0.24	1.20	0.19	0.95
0.10	2.65	0.35	0.018	0.42	1.20	0.26	0.74
0.10	2.65	0.50	0.025	0.59	1.18	0.32	0.64
0.10	2.65	0.60	0.030	0.67	1.12	0.36	0.60
0.25	1.31	0.25	0.031	0.19	0.76	0.12	0.48
0.25	1.31	0.35	0.044	0.27	0.77	0.13	0.37
0.25	1.31	0.45	0.056	0.33	0.73	0.14	0.31
0.25	1.31	0.55	0.069	0.37	0.67	0.15	0.27
0.40	0.99	0.30	0.060	0.16	0.53	0.09	0.30
0.40	0.99	0.40	0.080	0.22	0.55	0.10	0.25
0.40	0.99	0.45	0.090	0.23	0.51	0.12	0.27

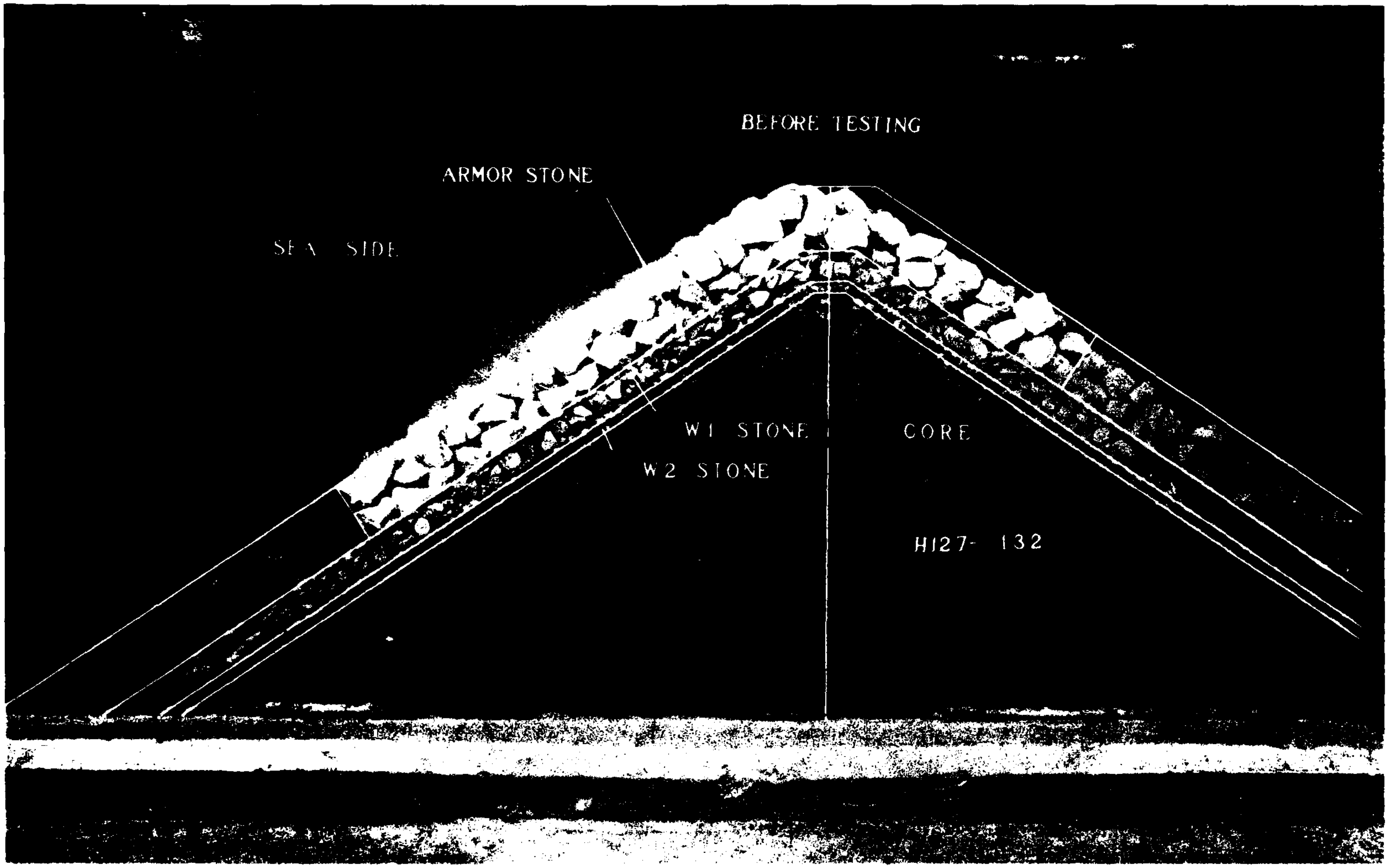


Photo 1. End view of typical test section before wave attack at a 1:1.5 sea-side structure slope

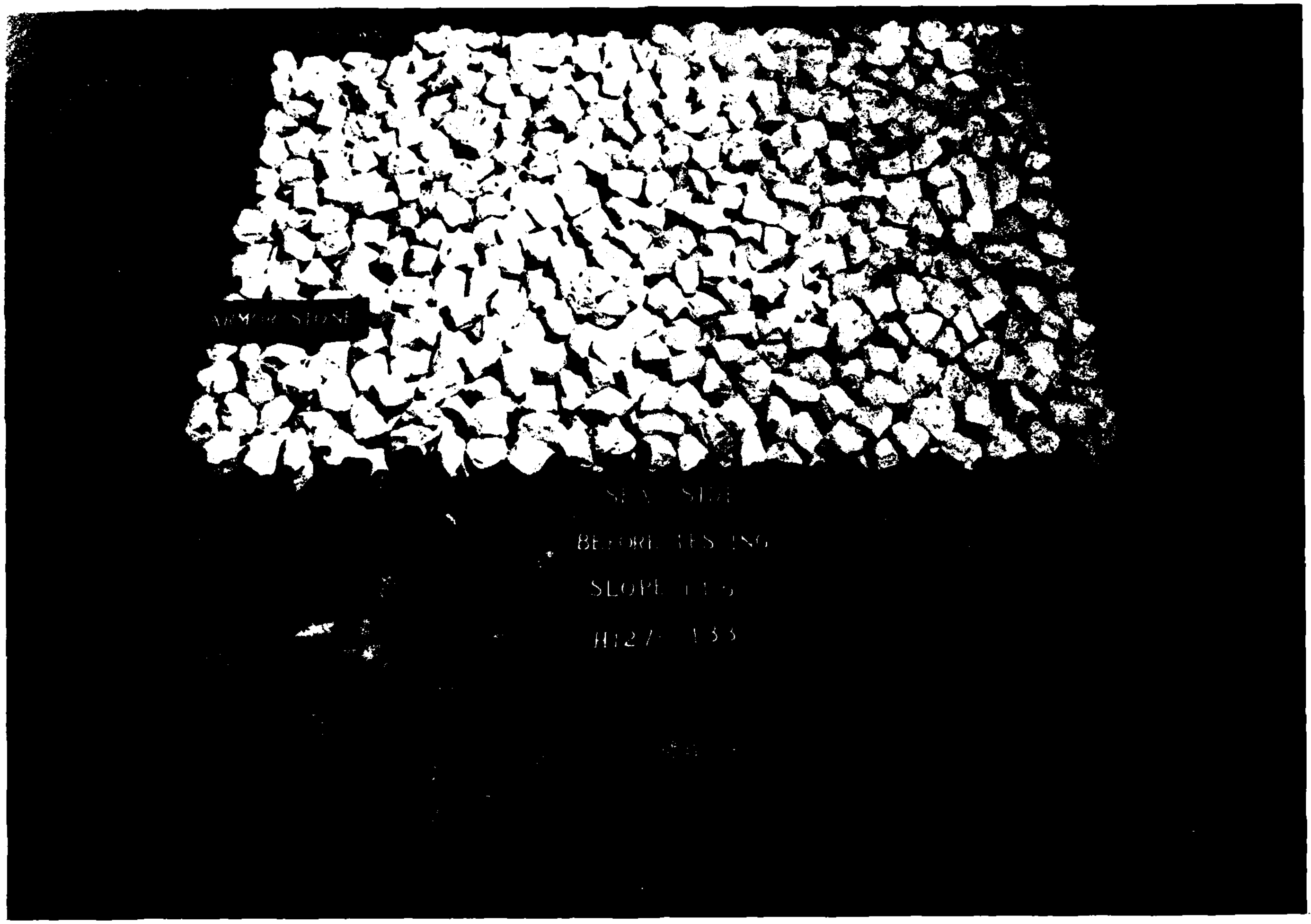


Photo 2. Sea-side view of typical test section before wave attack at a 1:1.5 sea-side structure slope

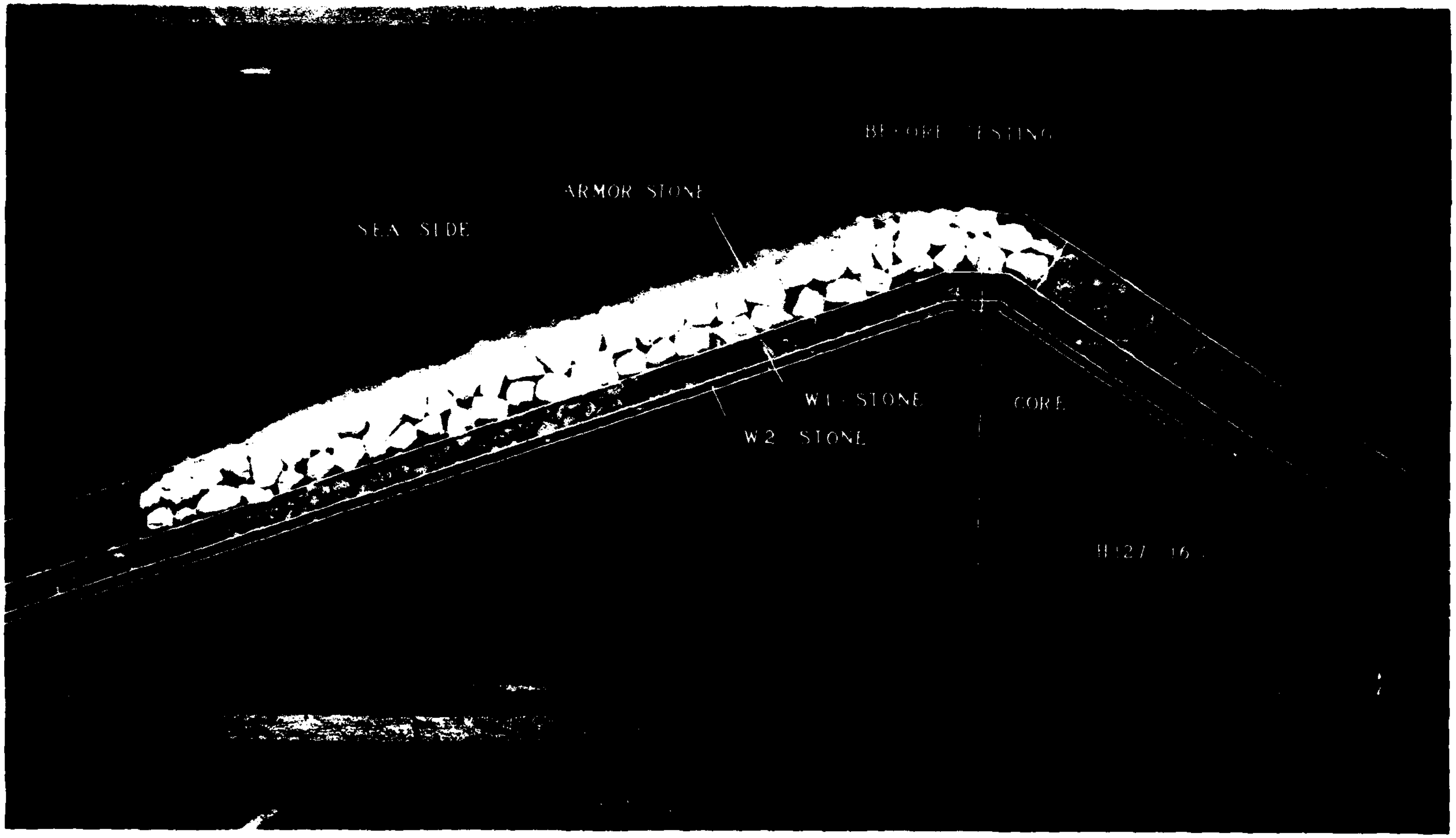


Photo 3. End view of typical test section before wave attack at a 1:3 sea-side structure slope

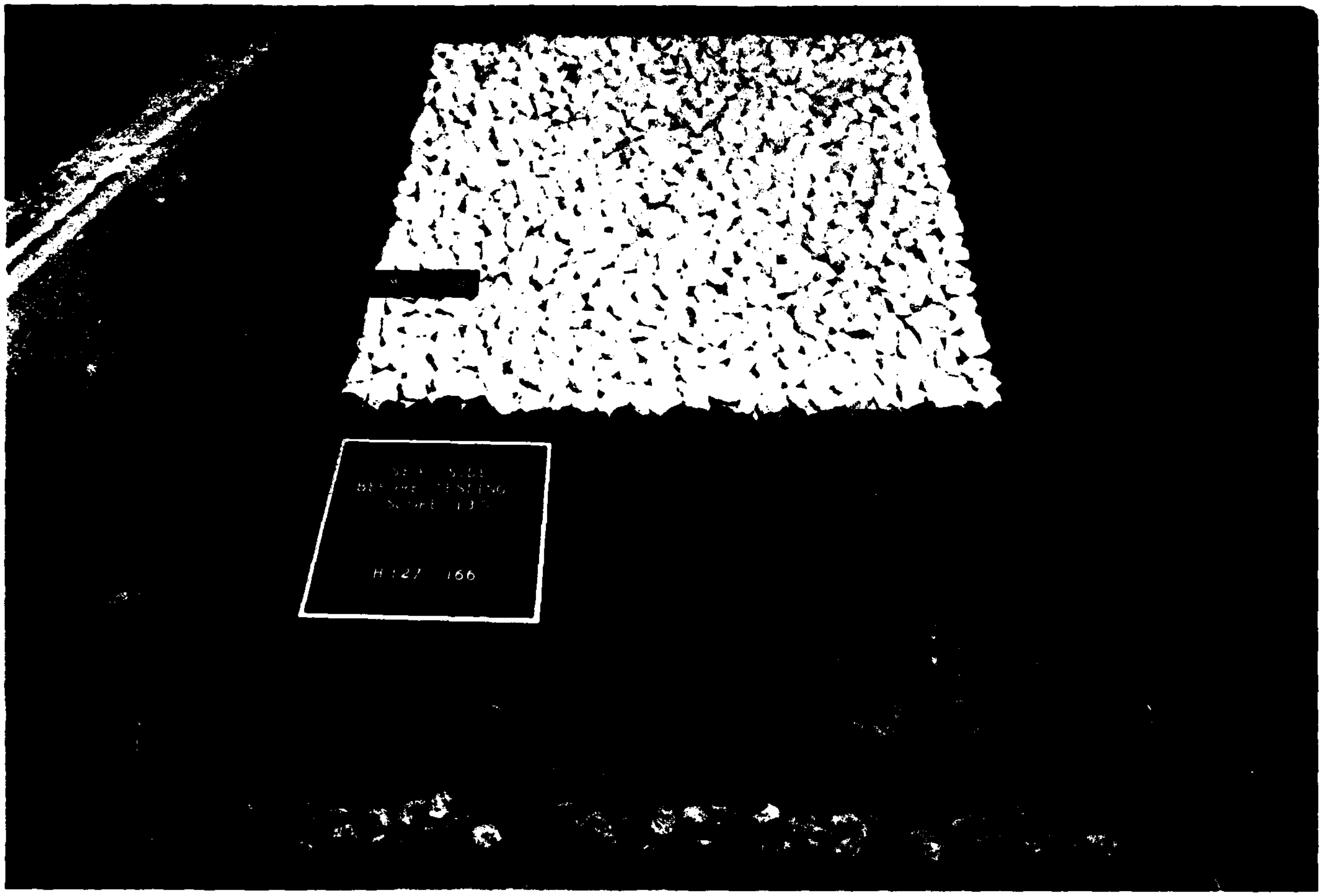


Photo 4. Sea-side view of typical test section before wave attack at a 1:3 sea-side structure slope

Photo 5. End view of typical test section under attack of 2.65-sec, 0.50-ft waves at a 1:1.5 sea-side structure slope; $W_1 = W_r/5$

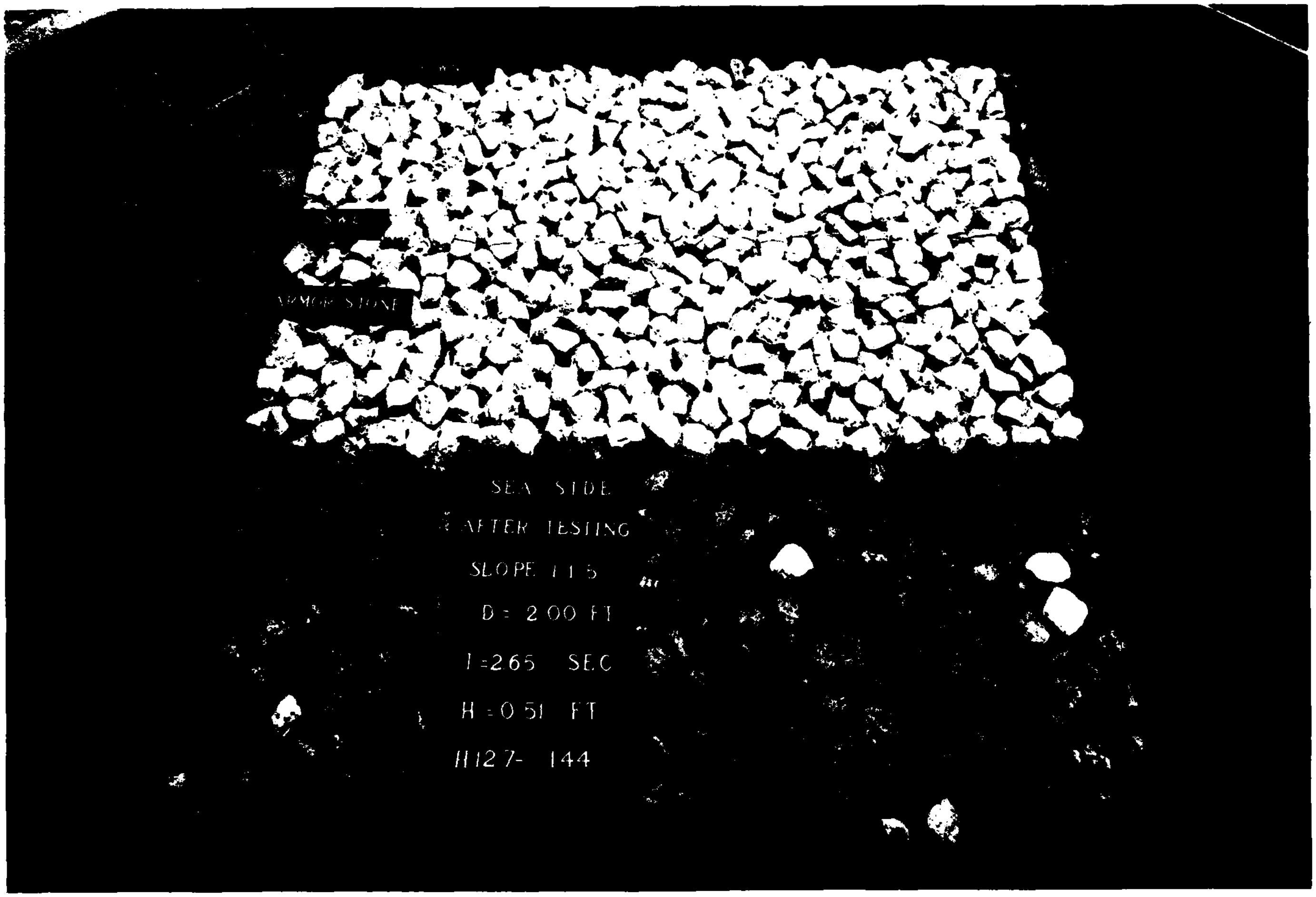


Photo 6. Sea-side view after attack of 2.65-sec, 0.51-ft waves; $W_1 = W_r/5$

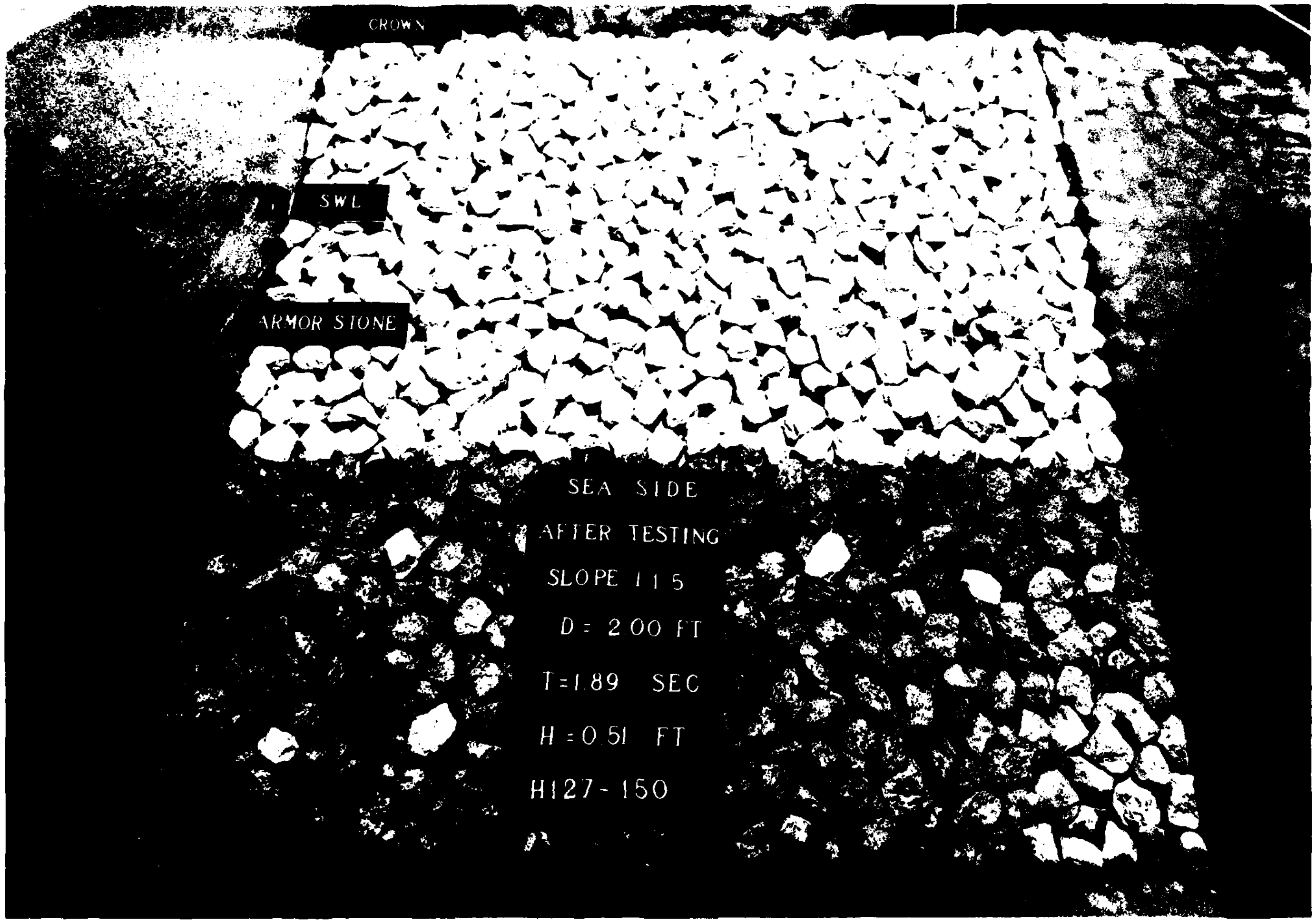


Photo 7. Sea-side view after attack of 1.89-sec, 0.51-ft waves; $W_1 = W_r/5$

Photo 8. Sea-side view after attack of 1.31-sec, 0.51-ft waves; $W_1 = W_r/5$

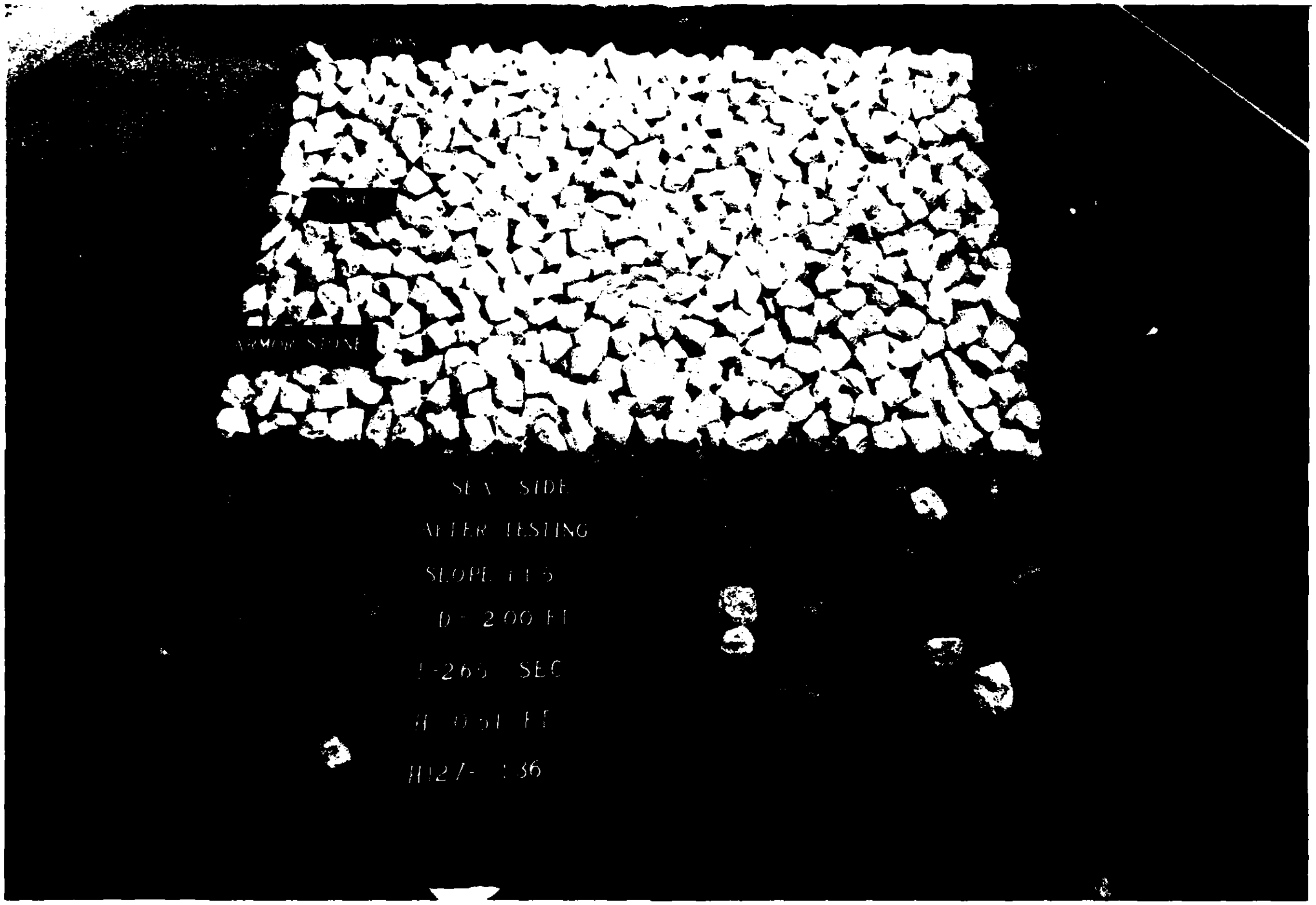


Photo 9. Sea-side view after attack of 2.65-sec, 0.51-ft waves; $W_1 = W_r/10$

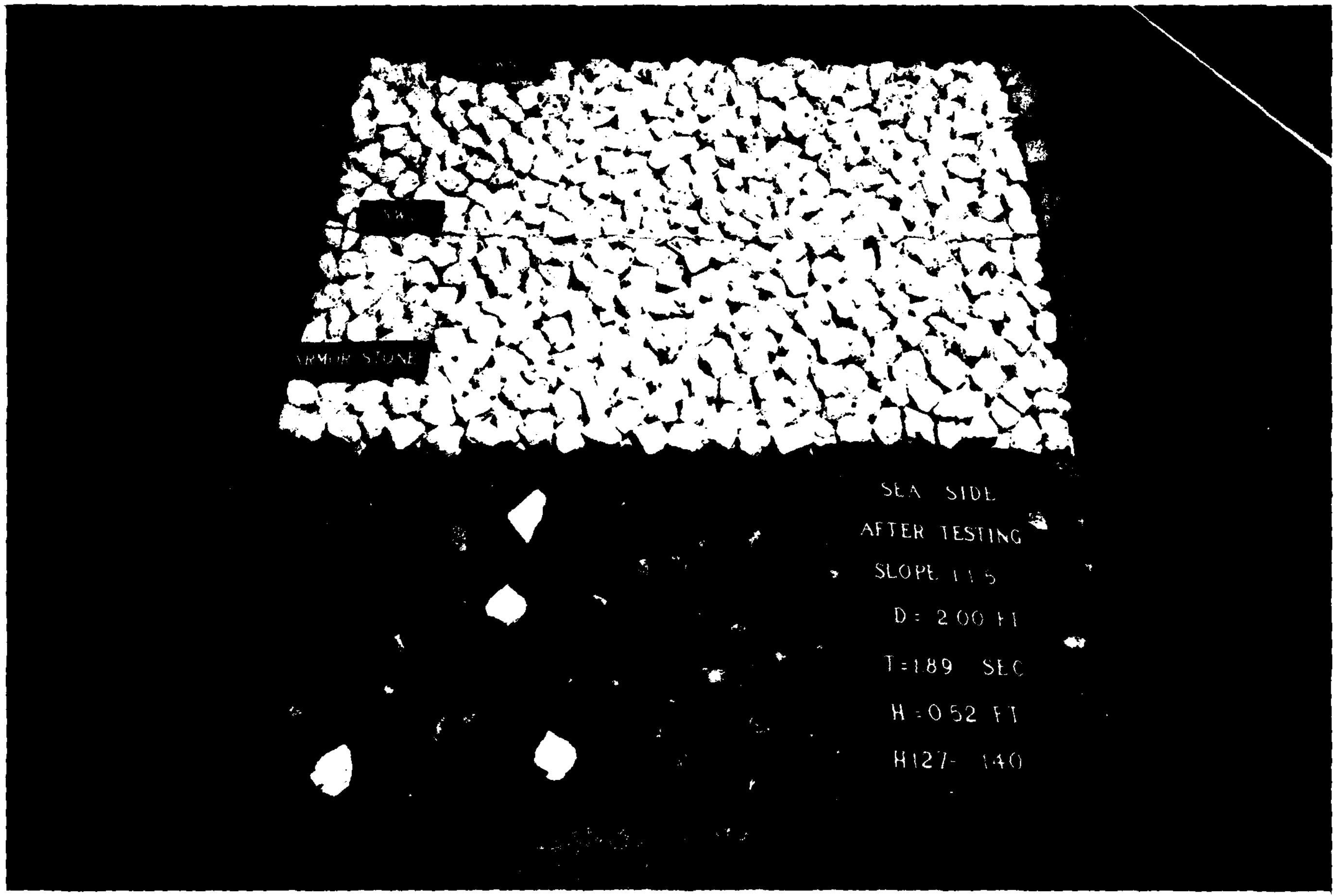


Photo 10. Sea-side view after attack of 1.89-sec, 0.52-ft waves; $W_1 = W_r/10$

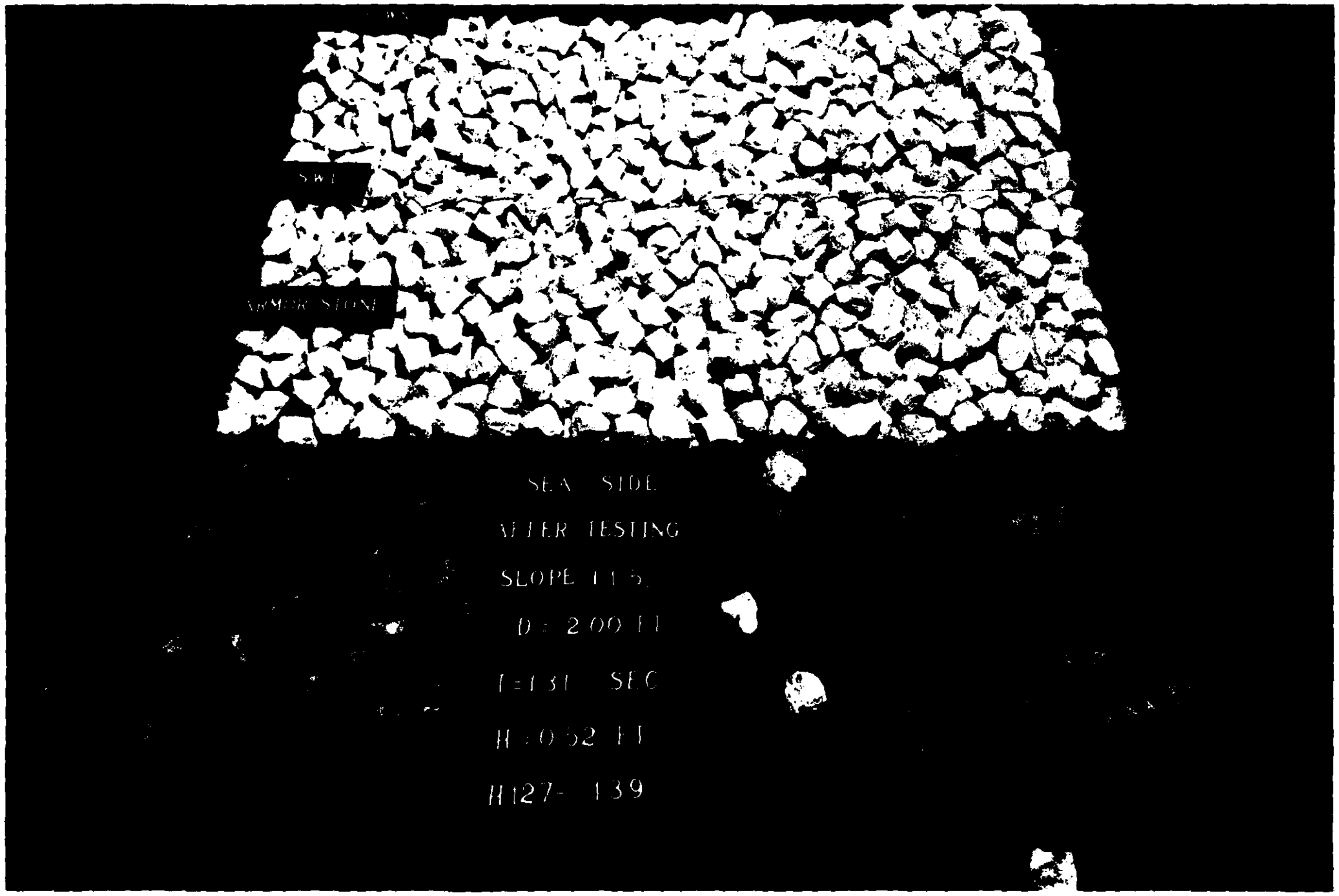


Photo 11. Sea-side view after attack of 1.31-sec, 0.52-ft waves; $W_1 = W_r/10$

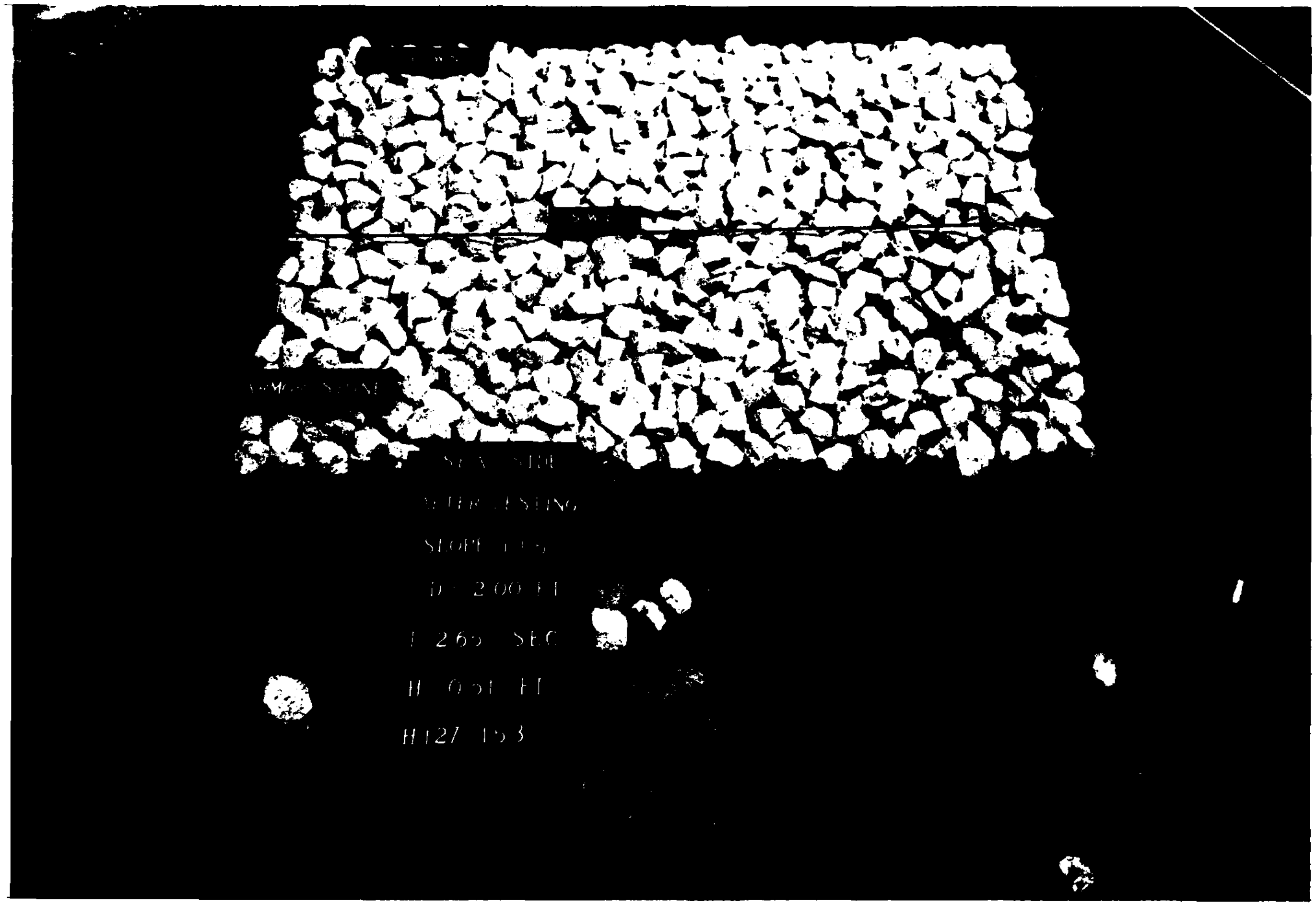


Photo 12. Sea-side view after attack of 2.65-sec, 0.51-ft waves; $W_1 = W_r/20$

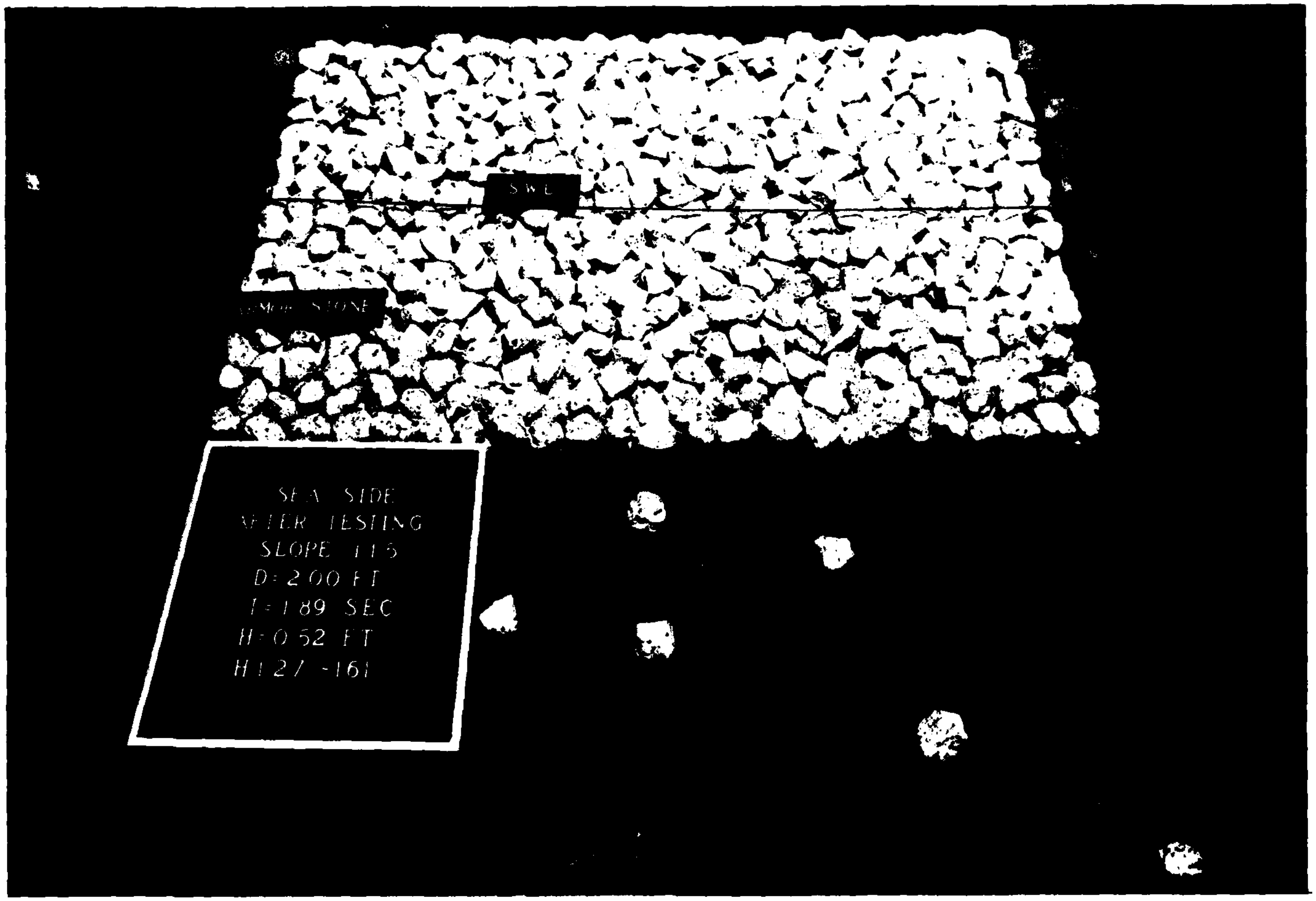


Photo 13. Sea-side view after attack of 1.89-sec, 0.52-ft waves; $W_1 = W_r/20$

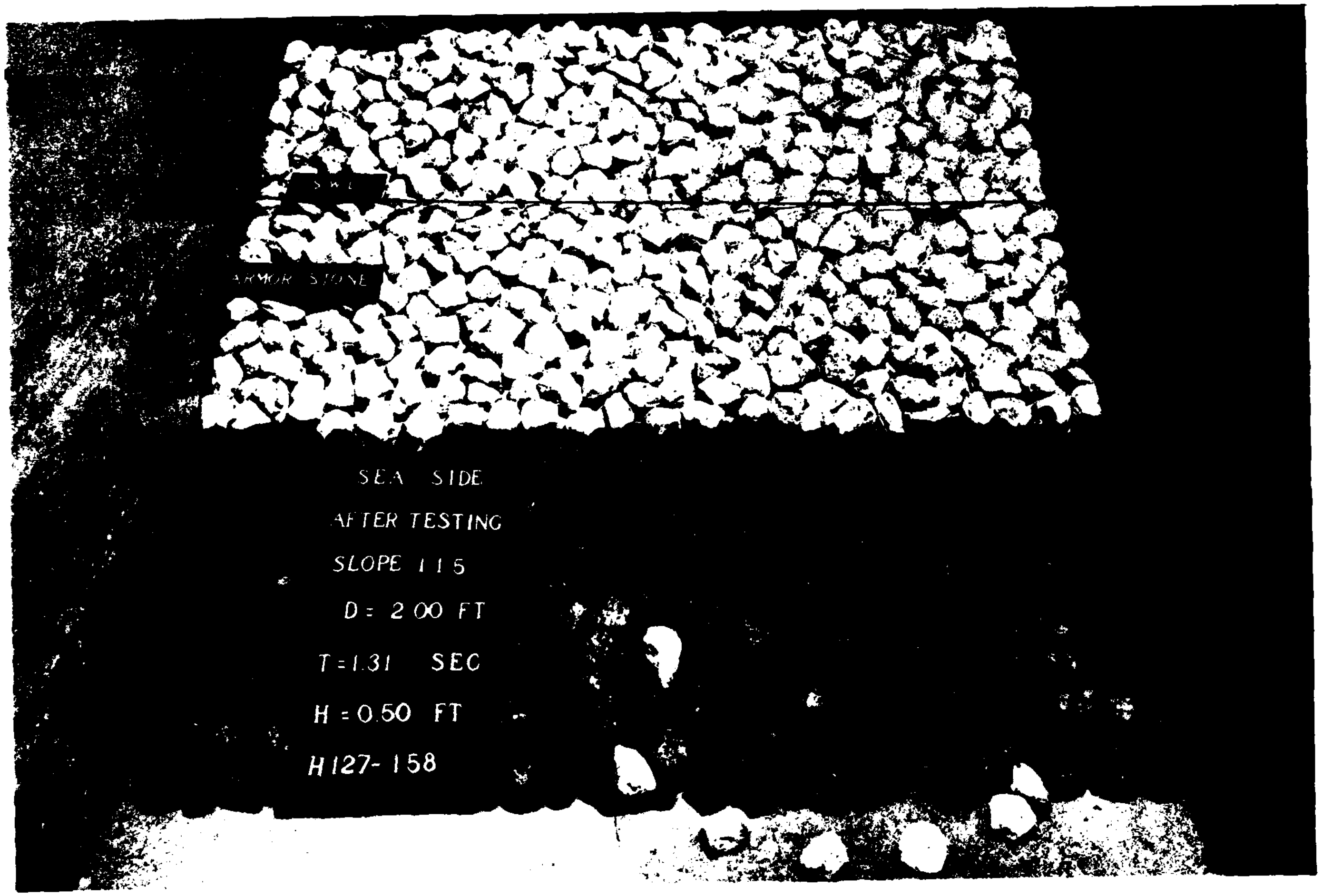


Photo 14. Sea-side view after attack of 1.31-sec, 0.50-ft waves; $W_1 = W_r/20$

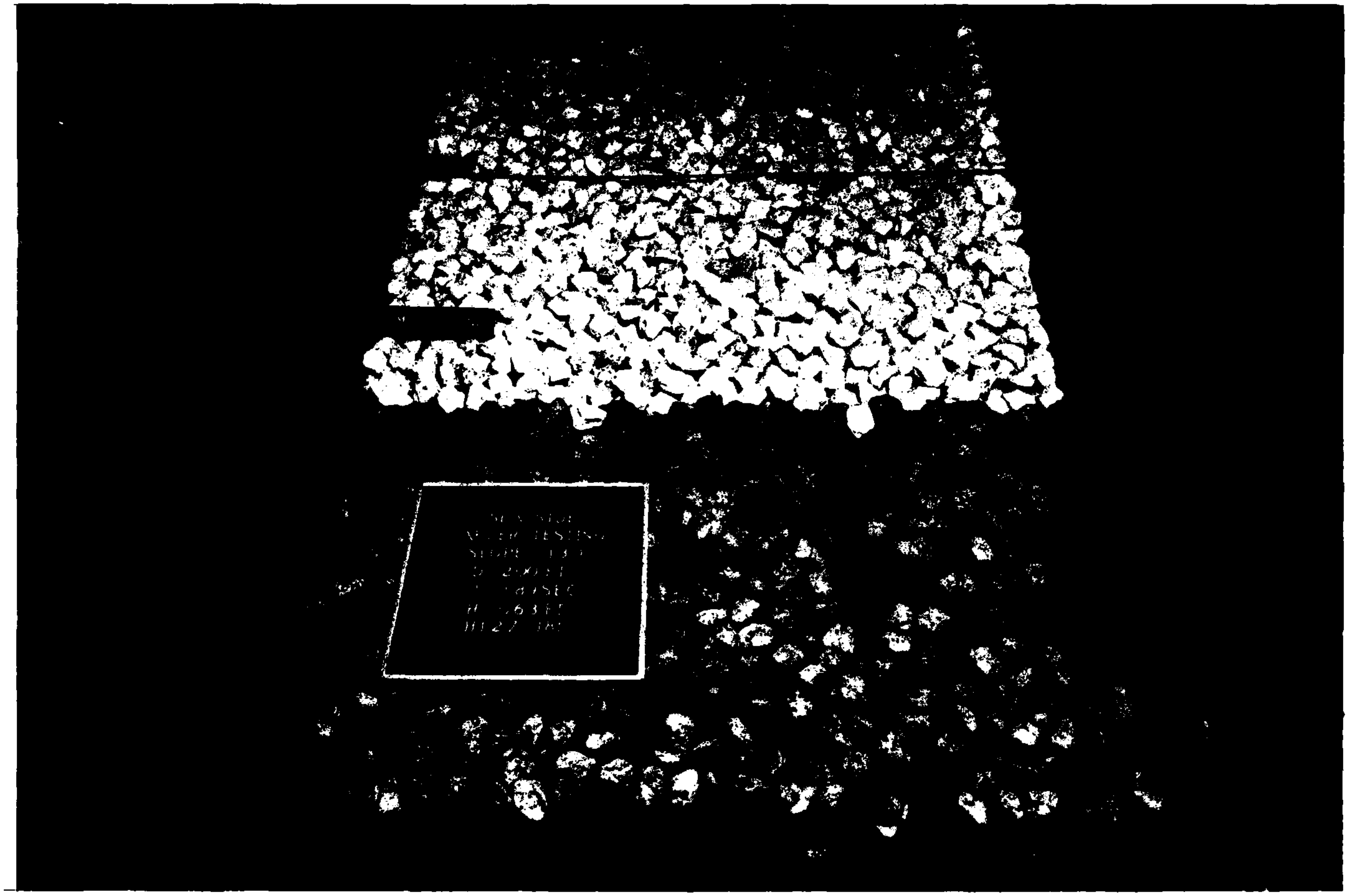


Photo 15. Sea-side view after attack of 1.89-sec, 0.63-ft waves; $W_1 = W_r/5$

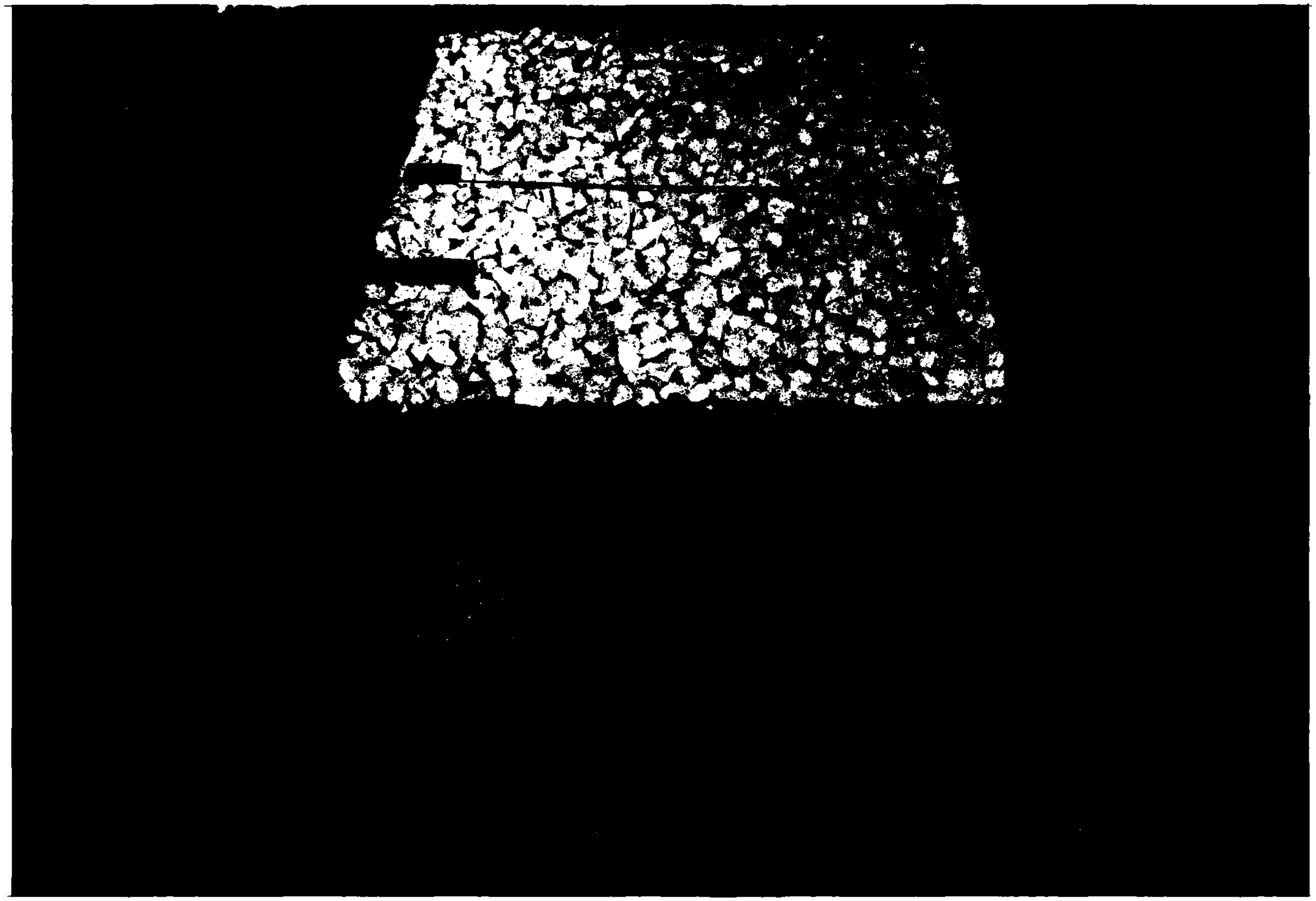


Photo 16. Sea-side view after attack of 1.52-sec, 0.63-ft waves; $W_1 = W_r/5$

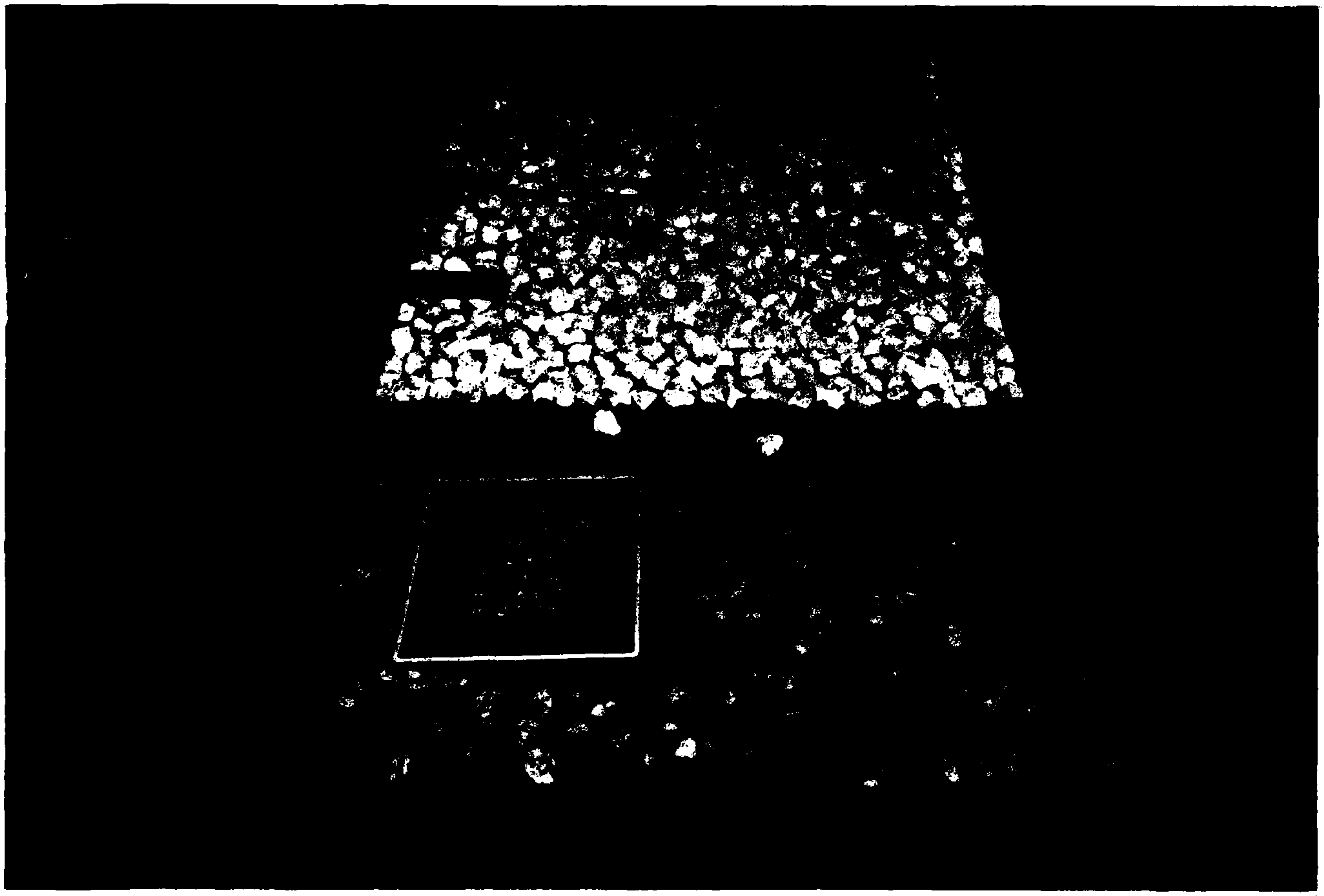


Photo 17. Sea-side view after attack of 1.31-sec, 0.63-ft waves; $W_1 = W_r/5$

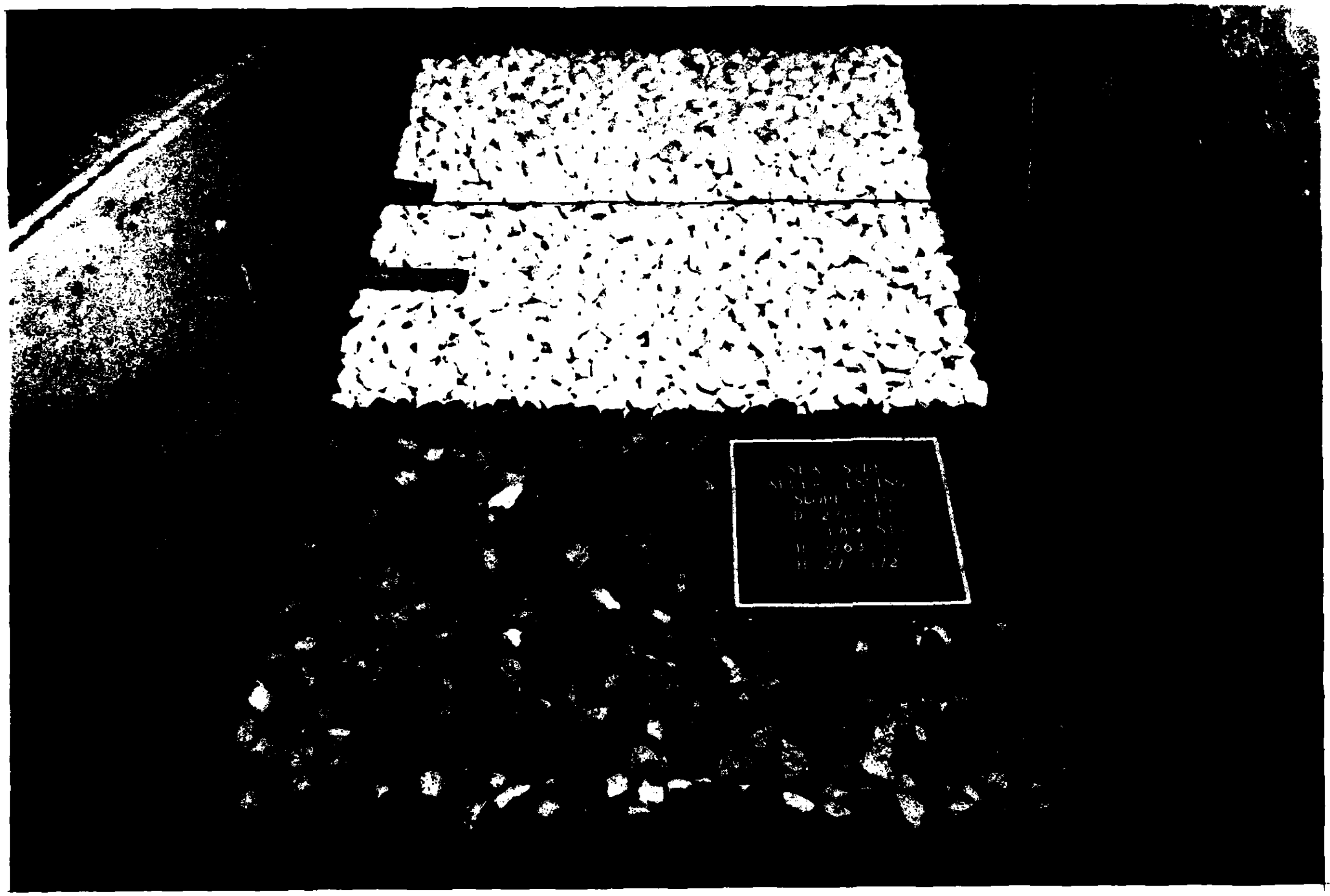


Photo 18. Sea-side view after attack of 1.89-sec, 0.63-ft waves; $W_1 = W_r/10$

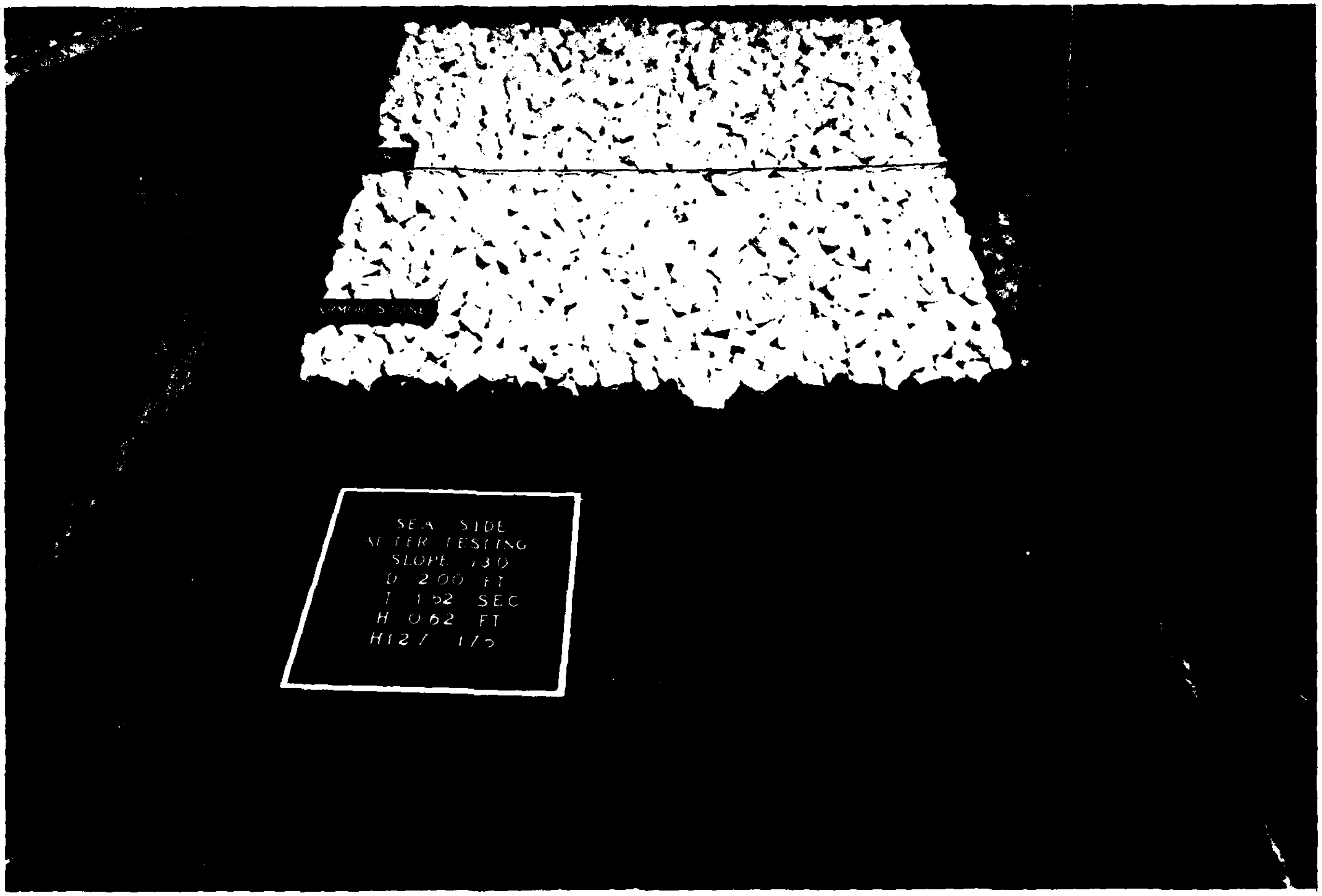


Photo 19. Sea-side view after attack of 1.52-sec, 0.62-ft waves; $W_1 = W_r/10$

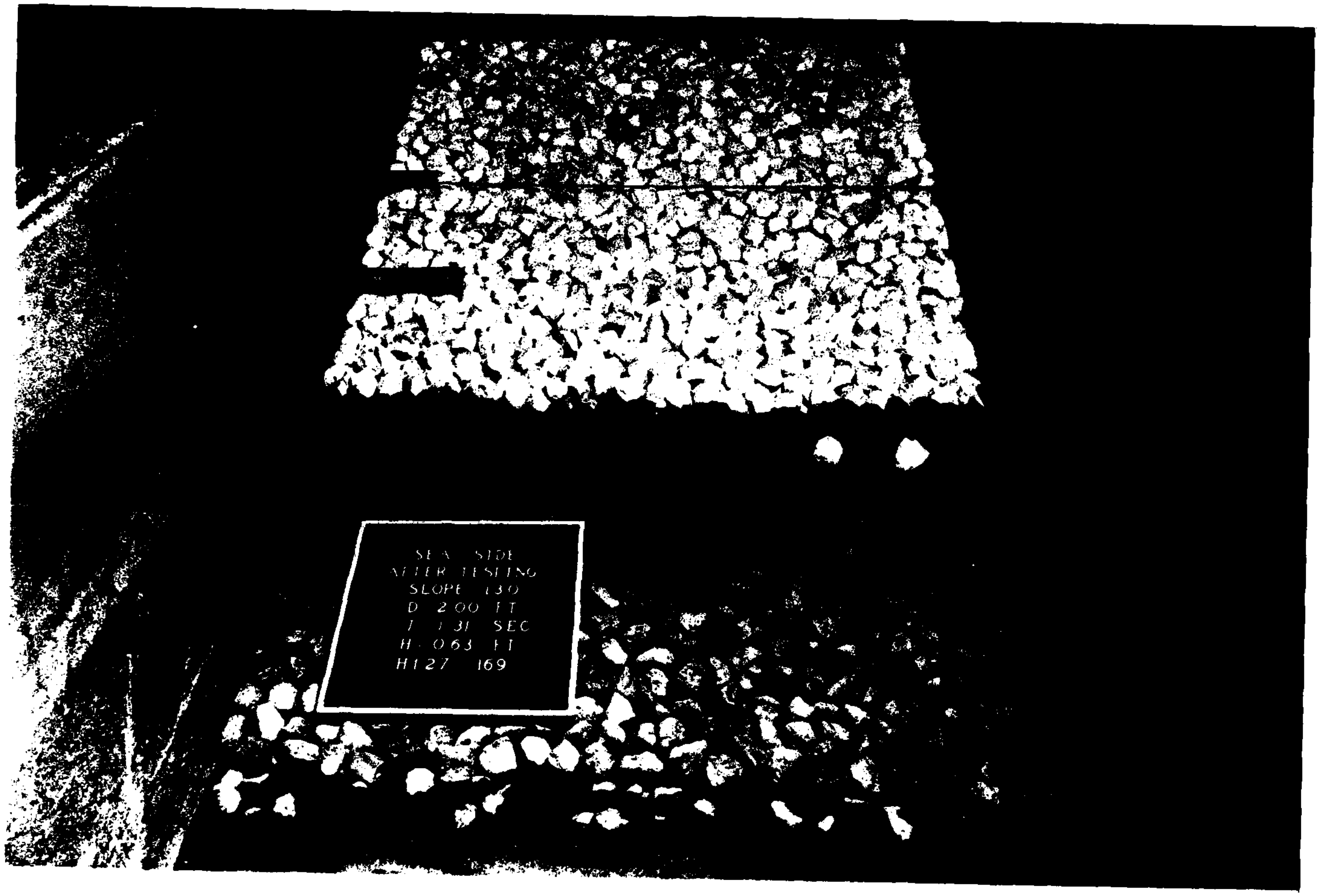


Photo 20. Sea-side view after attack of 1.31-sec, 0.63-ft waves; $W_1 = W_r/10$

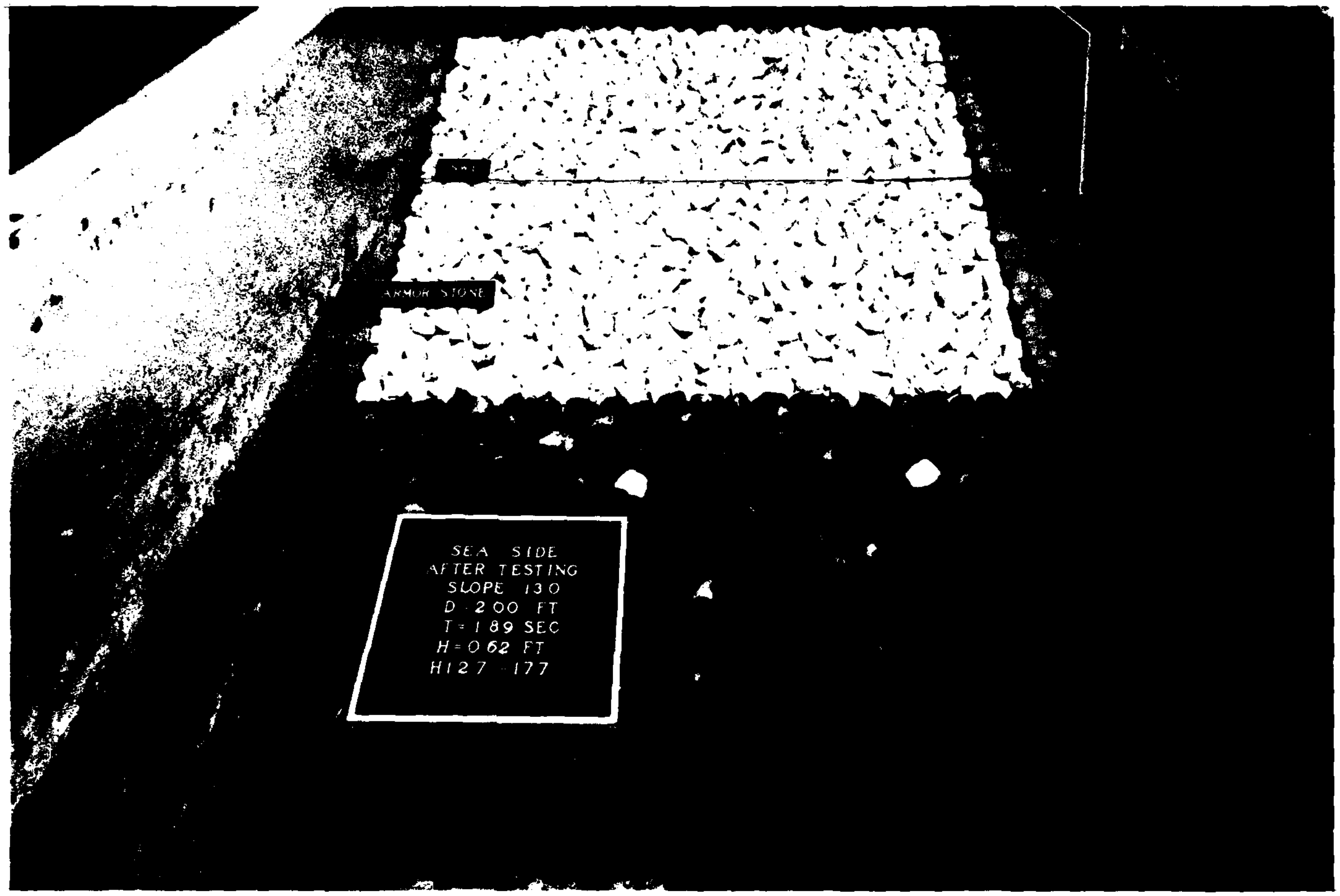


Photo 21. Sea-side view after attack of 1.89-sec, 0.62-ft waves; $W_1 = W_r/20$

Photo 22. Sea-side view after attack of 1.52-sec, 0.62-ft waves; $W_1 = W_r/20$

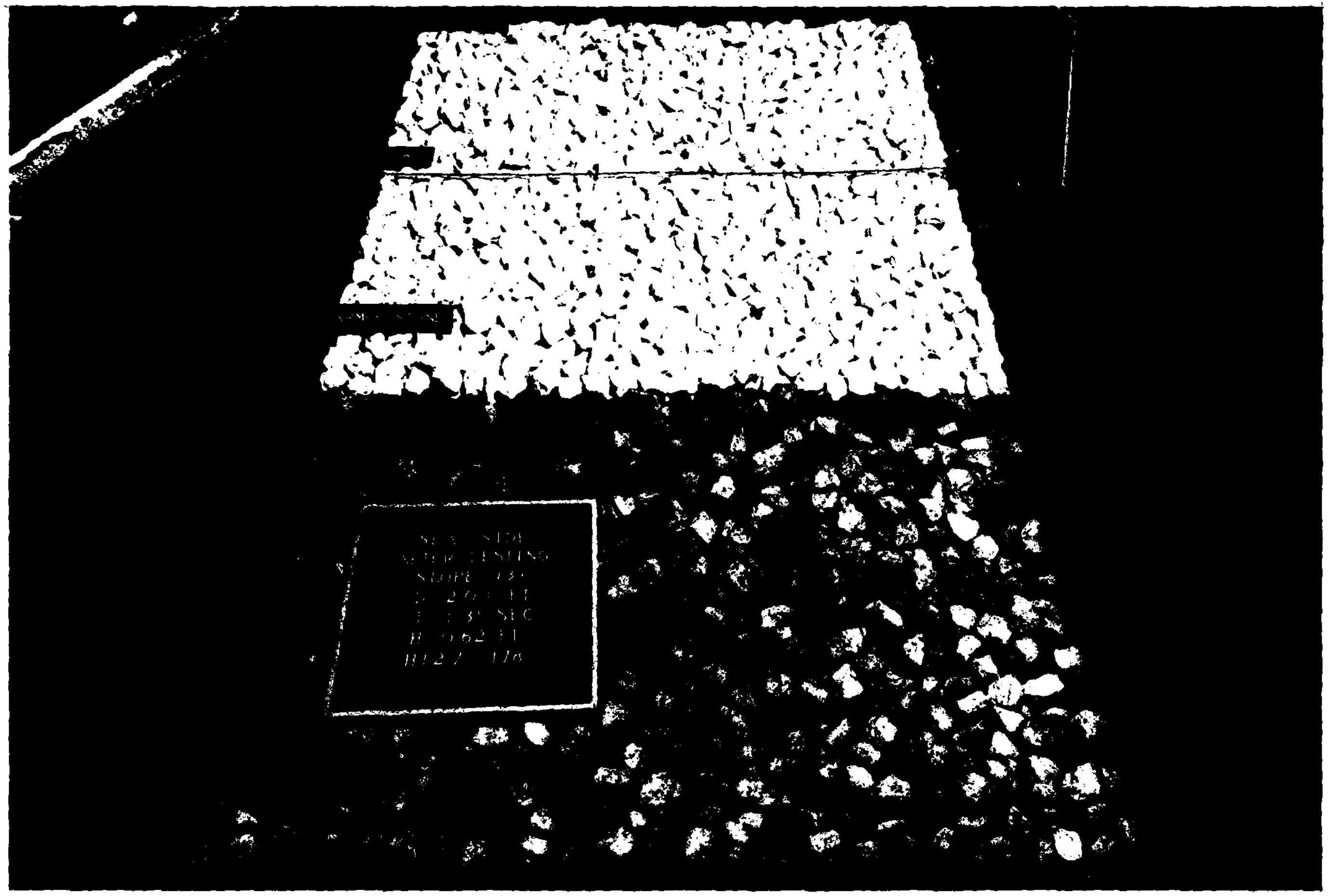
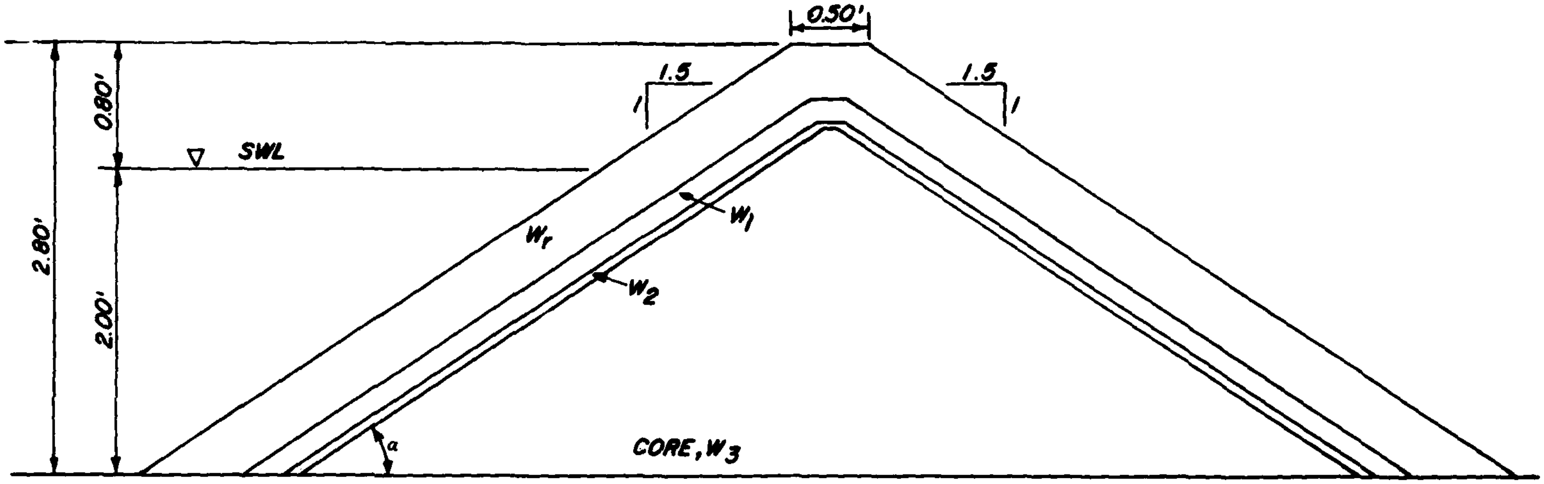


Photo 23. Sea-side view after attack of 1.31-sec, 0.62-ft waves; $W_1 = W_r/20$

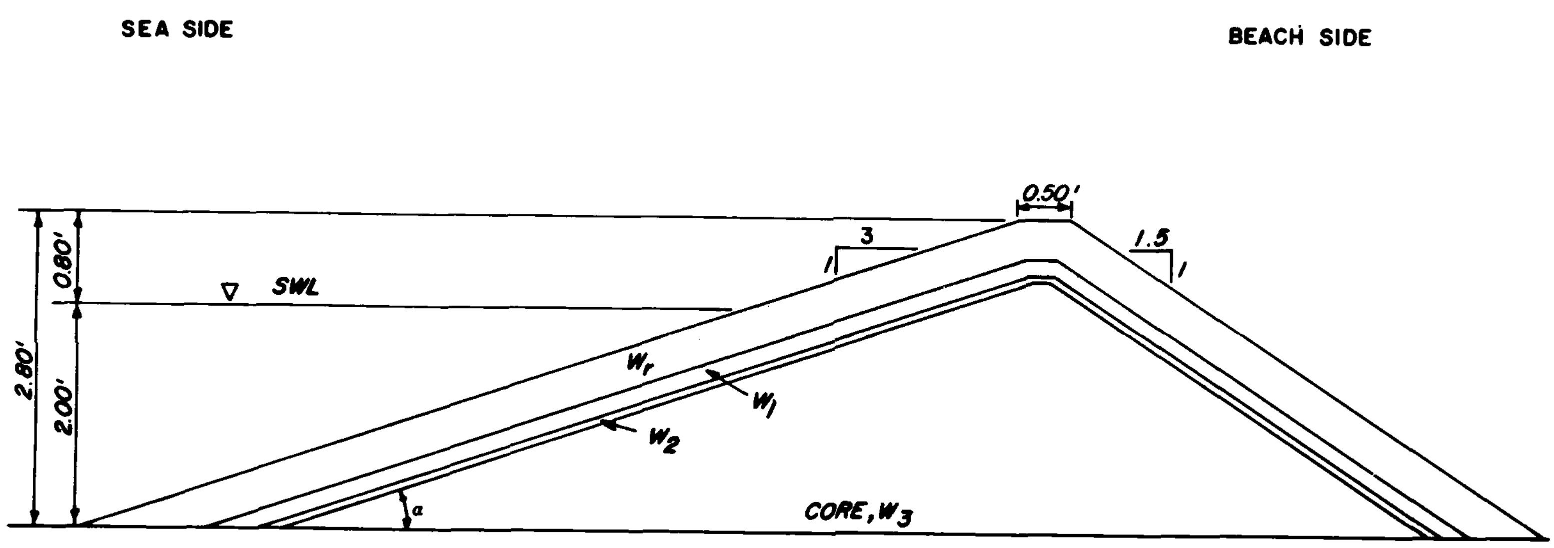


MATERIAL WEIGHTS, LB

MATERIAL	$W_1 = W_r/5$	$W_1 = W_r/10$	$W_1 = W_r/20$
W_r	0.55	0.55	0.55
W_1	0.11	0.055	0.028
W_2	0.0055	0.0028	0.0014
W_3	← 0.0011 TO 0.00018 →		

TYPICAL BREAKWATER CROSS SECTION

$\cot \alpha = 1.5$

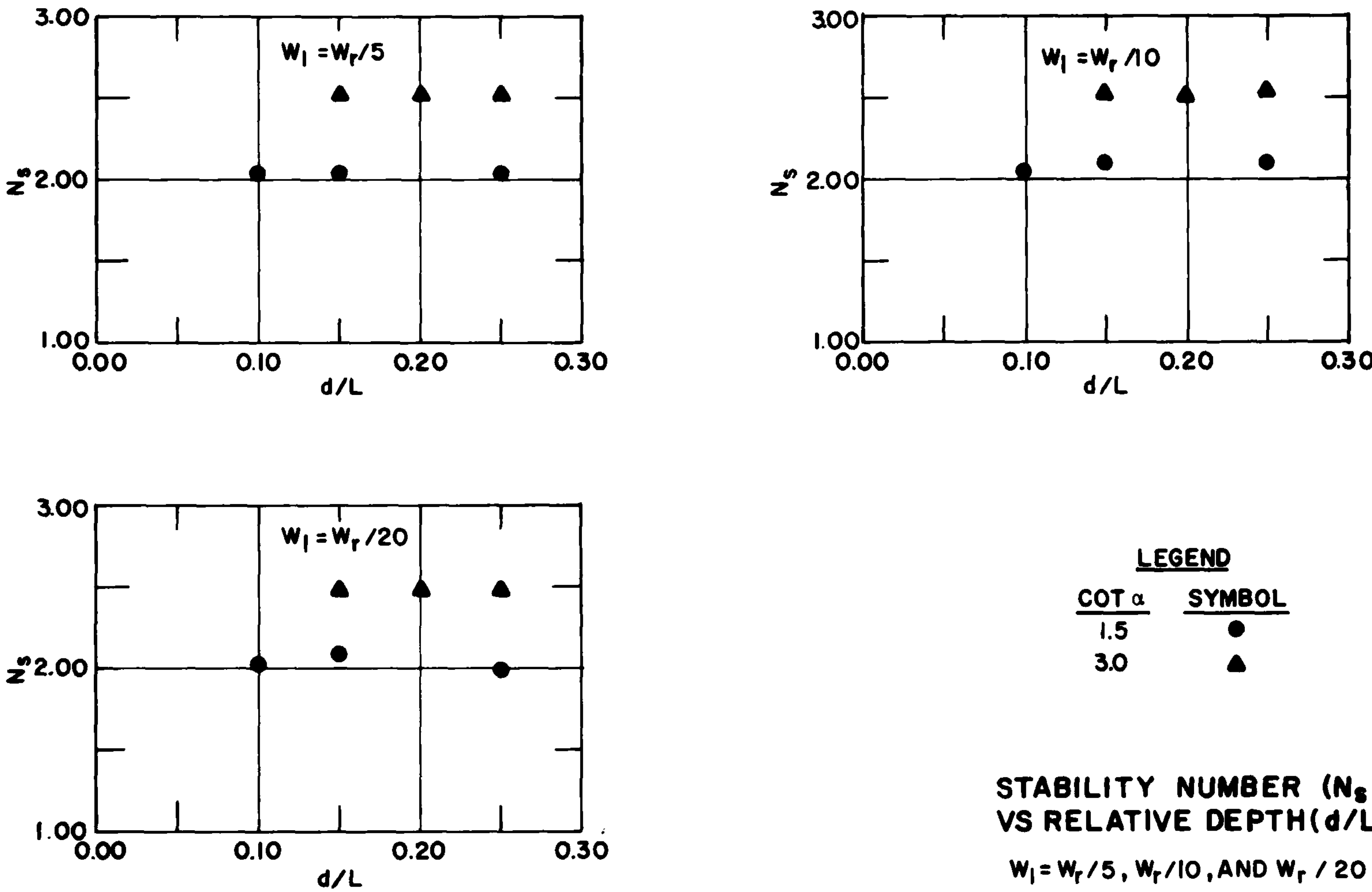


MATERIAL WEIGHTS, LB

MATERIAL	$W_1 = W_r/5$	$W_1 = W_r/10$	$W_1 = W_r/20$
W_r	0.55	0.55	0.55
W_1	0.11	0.035	0.028
W_2	0.0055	0.0028	0.0014
W_3	← 0.0011 TO 0.00018 →		

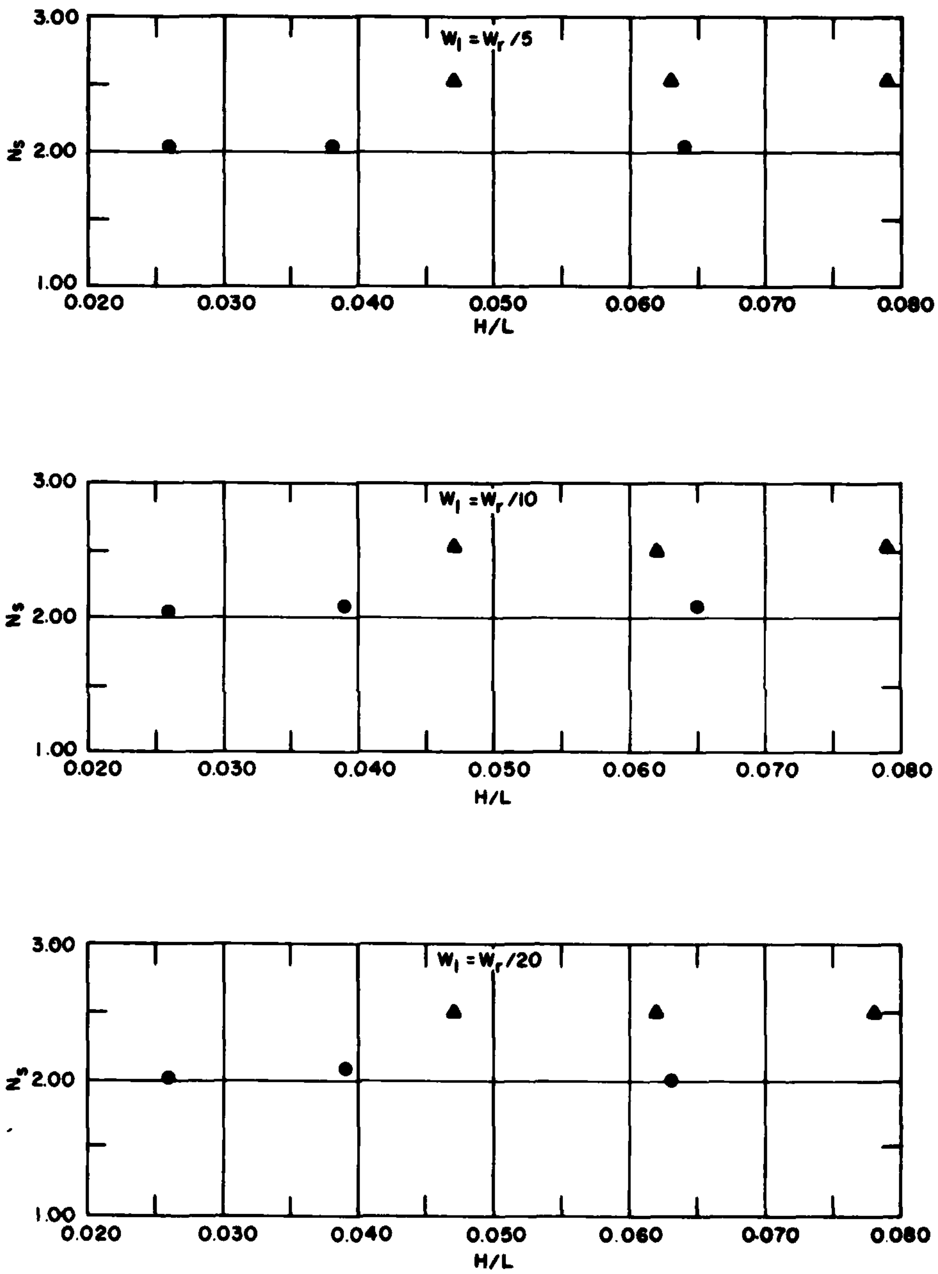
TYPICAL BREAKWATER
CROSS SECTION

$\cot\alpha = 3.0$



STABILITY NUMBER (N_s)
VS RELATIVE DEPTH (d/L)

$W_l = W_r/5$, $W_r/10$, AND $W_r/20$



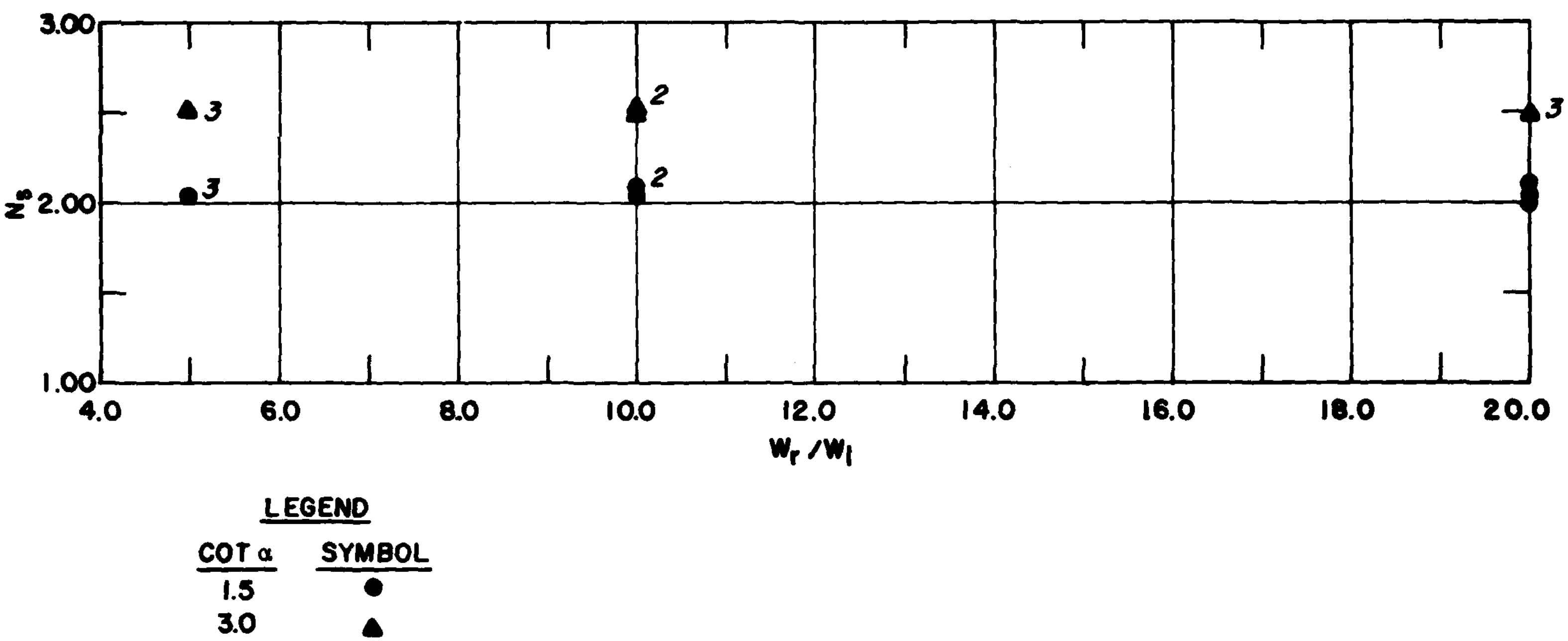
LEGEND

COT α	SYMBOL
1.5	●
3.0	▲

STABILITY NUMBER (N_s)
VS WAVE STEEPNESS (H/L)

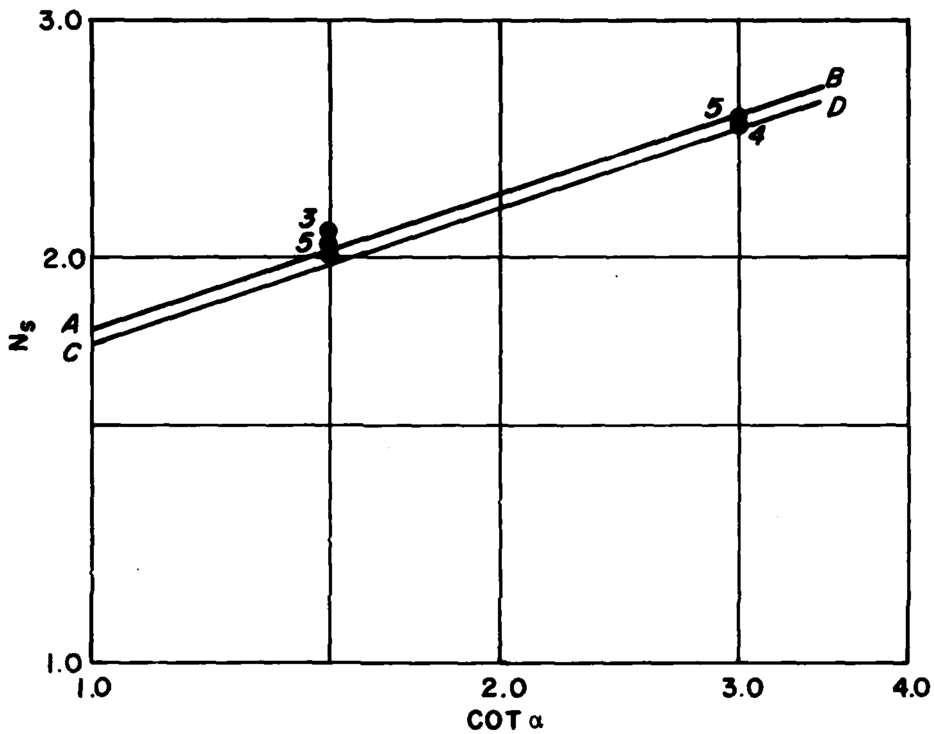
$W_l = W_r/5$, $W_r/10$, AND $W_r/20$

PLATE 4



NOTE: NUMBERS BESIDE DATA POINTS INDICATE THAT THE NUMBER OF DATA POINTS EXCEEDS ONE

STABILITY NUMBER (N_s) VS RELATIVE UNDERLAYER WEIGHT (W_r / W_l)



NOTES: LINE AB CORRESPONDS TO K = 5.5 (AVERAGE)

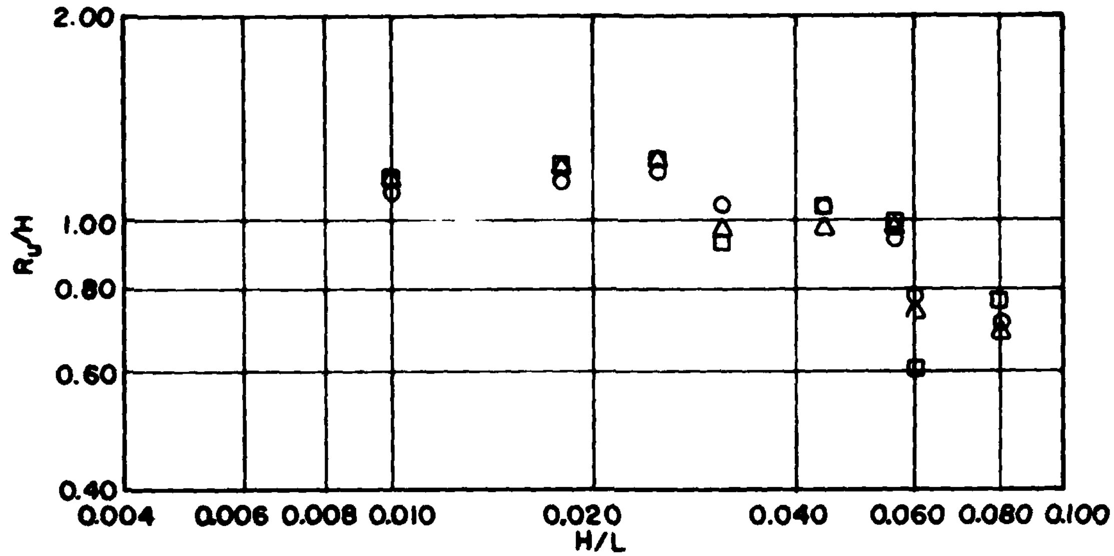
LINE CD CORRESPONDS TO K = 5.1 (LOWER LIMIT)

NUMBERS BESIDE DATA POINTS INDICATE THAT THE NUMBER OF DATA POINTS EXCEEDS ONE

$$N_s = \frac{\gamma_r^{1/3} H}{(S_r - 1) W_r^{1/3}}$$

STABILITY NUMBER (N_s)
VS COT α

PLATE 6

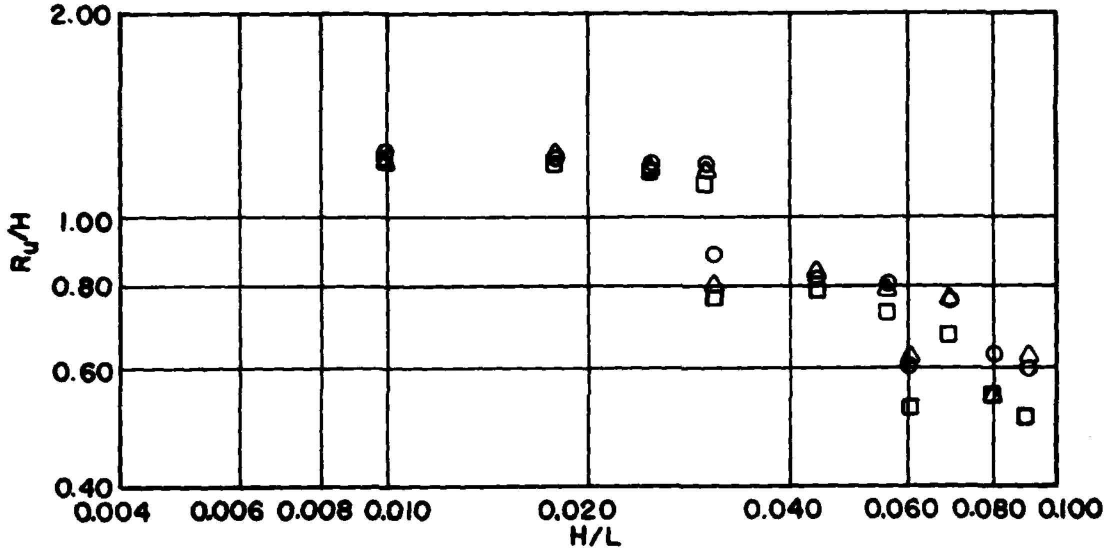


LEGEND

SYMBOL	W_1
○	$W_r/5$
△	$W_r/10$
□	$W_r/20$

COMPARATIVE VALUES OF RELATIVE RUNUP (R_u/H) VS WAVE STEEPNESS (H/L)

COT $\alpha = 1.5$
$W_1 = W_r/5$, $W_r/10$, AND $W_r/20$

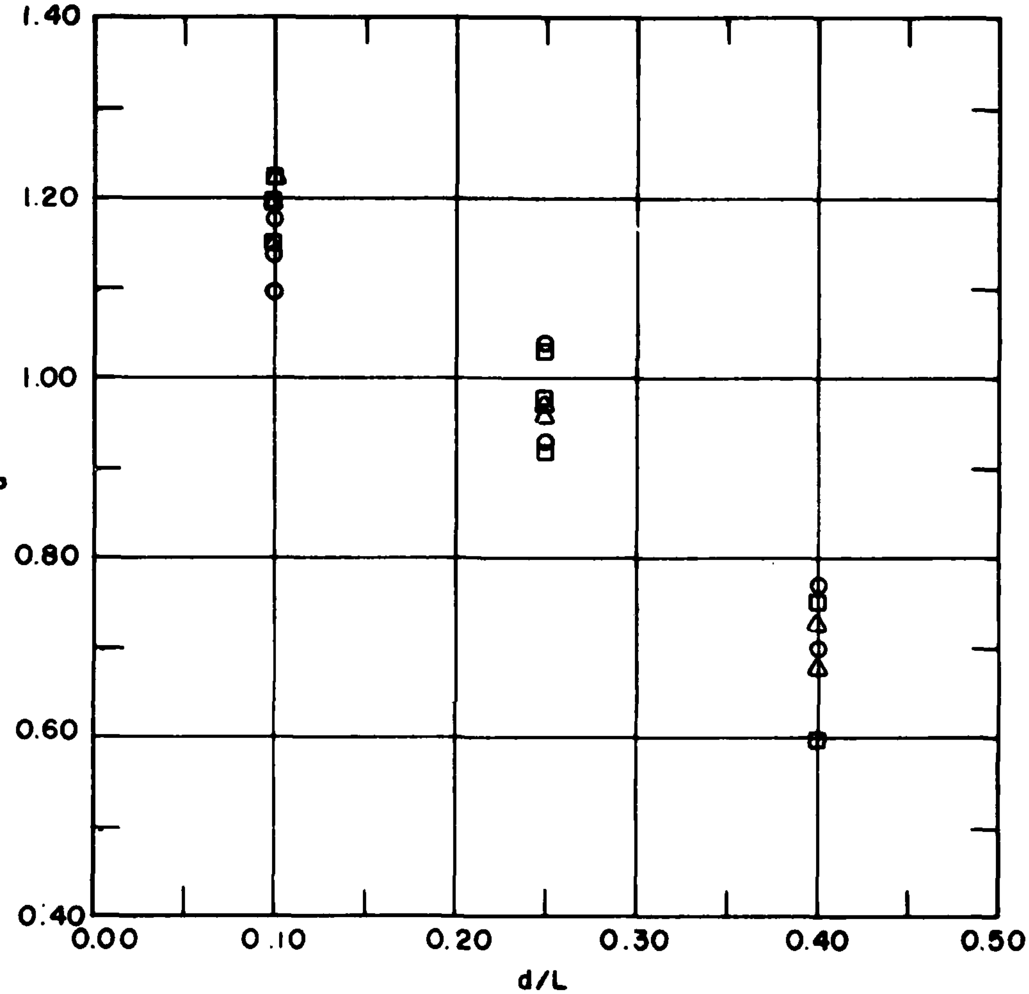


LEGEND

SYMBOL	W_l
○	$W_r/5$
△	$W_r/10$
□	$W_r/20$

COMPARATIVE VALUES OF RELATIVE RUNUP (R_u/H) VS WAVE STEEPNESS (H/L)

COT α = 3.0
$W_l = W_r/5$, $W_r/10$, AND $W_r/20$

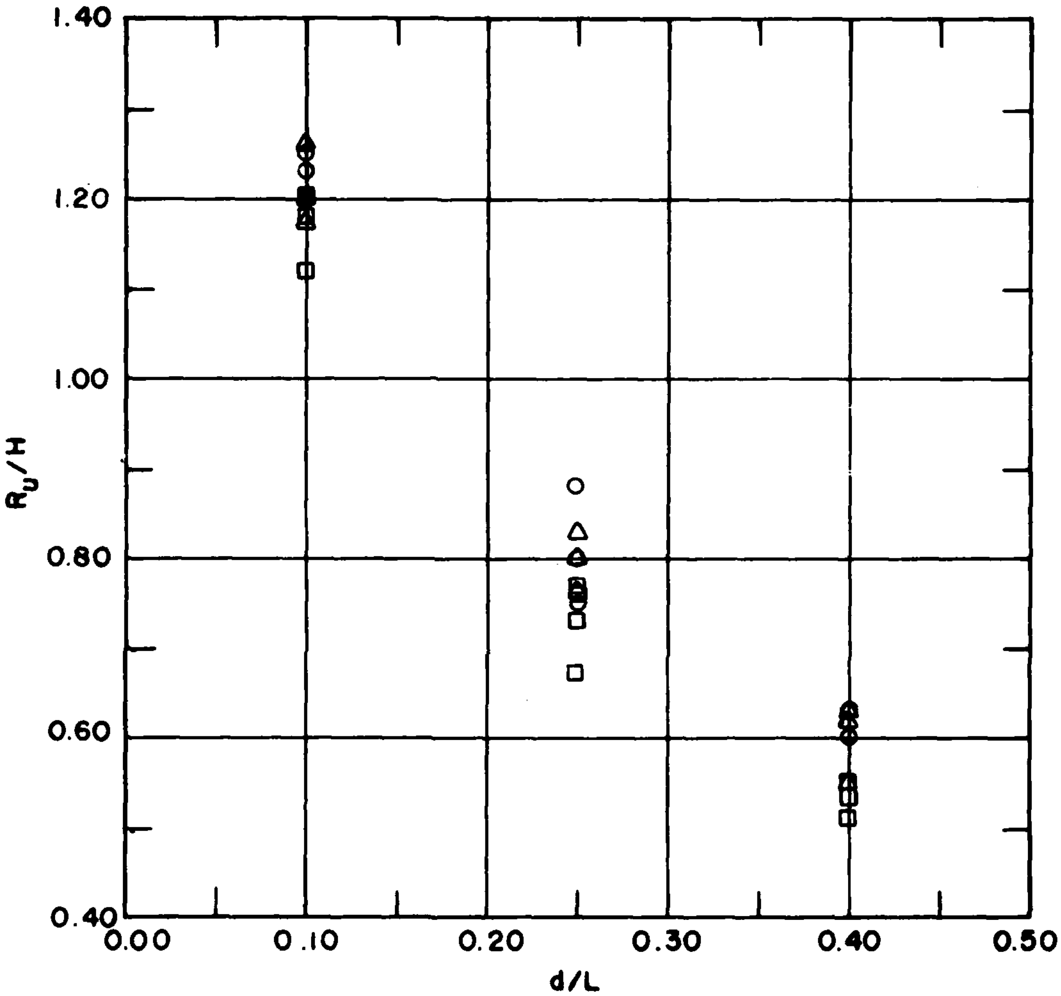


LEGEND

SYMBOL	W_1
O	$W_r/5$
Δ	$W_r/10$
□	$W_r/20$

COMPARATIVE VALUES OF RELATIVE RUNUP (R_u/H) VS RELATIVE DEPTH (d/L)

COT α = 1.5
$W_1 = W_r/5$, $W_r/10$, AND $W_r/20$

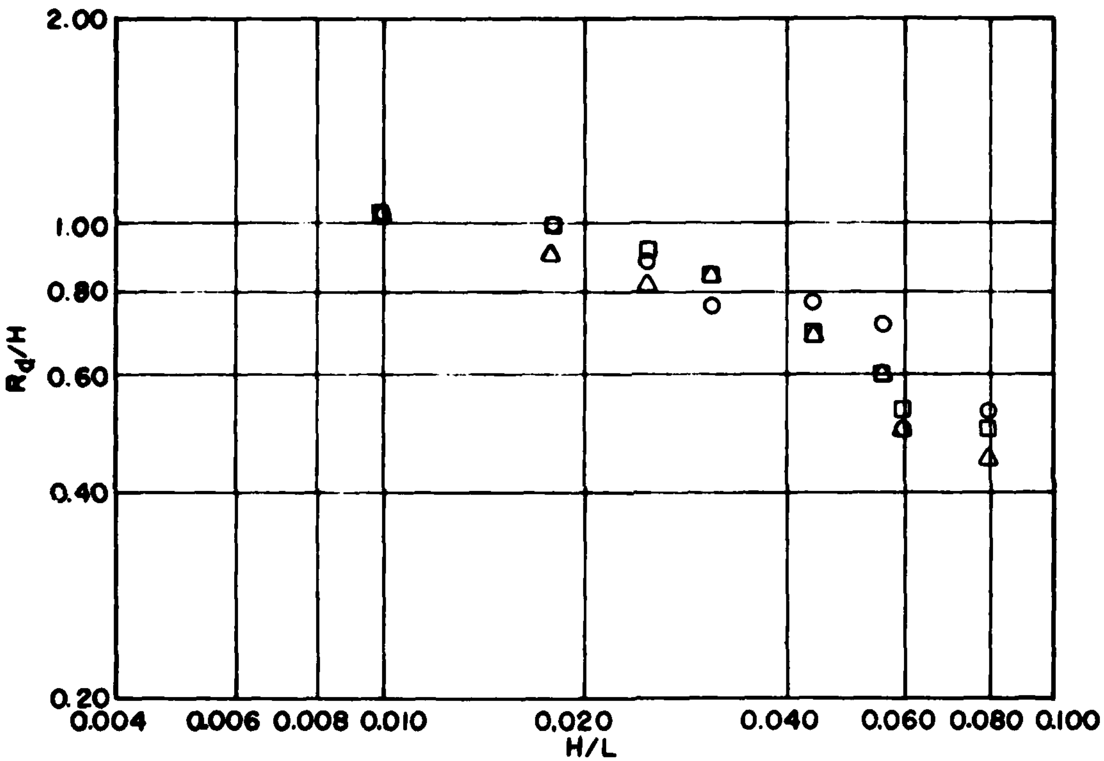


LEGEND

SYMBOL	W_l
○	$W_r/5$
△	$W_r/10$
□	$W_r/20$

COMPARATIVE VALUES OF RELATIVE RUNUP (R_u/H) VS RELATIVE DEPTH (d/L)

COT α = 3.0
$W_l = W_r/5$, $W_r/10$, AND $W_r/20$

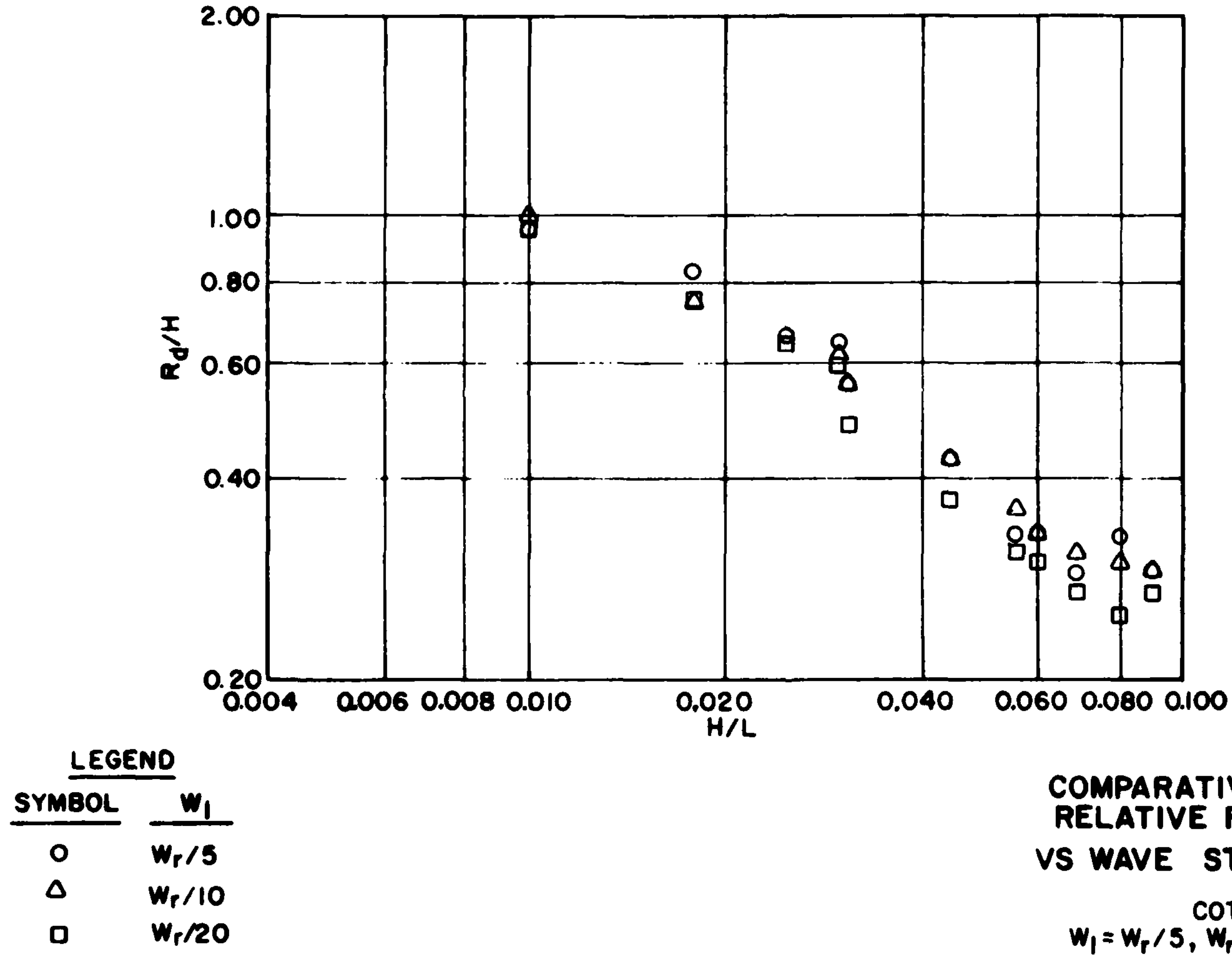


LEGEND

SYMBOL	W_l
○	$W_r/5$
△	$W_r/10$
□	$W_r/20$

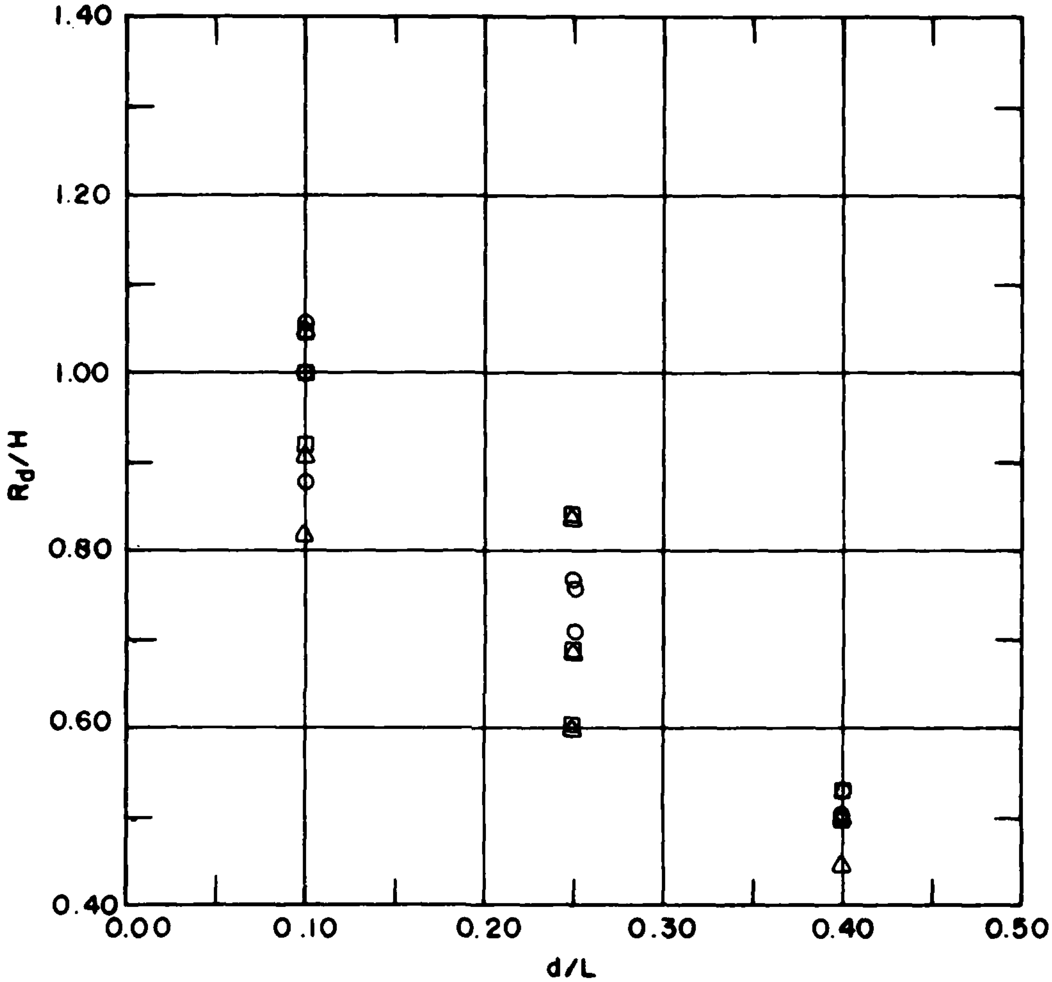
COMPARATIVE VALUES OF RELATIVE RUNDOWN (R_d/H) VS WAVE STEEPNESS (H/L)

COT α = 1.5
$W_l = W_r/5$, $W_r/10$, AND $W_r/20$



COMPARATIVE VALUES OF RELATIVE RUNDOWN (R_d/H) VS WAVE STEEPNESS (H/L)

COT α = 3.0
$W_1 = W_r/5$, $W_r/10$, AND $W_r/20$

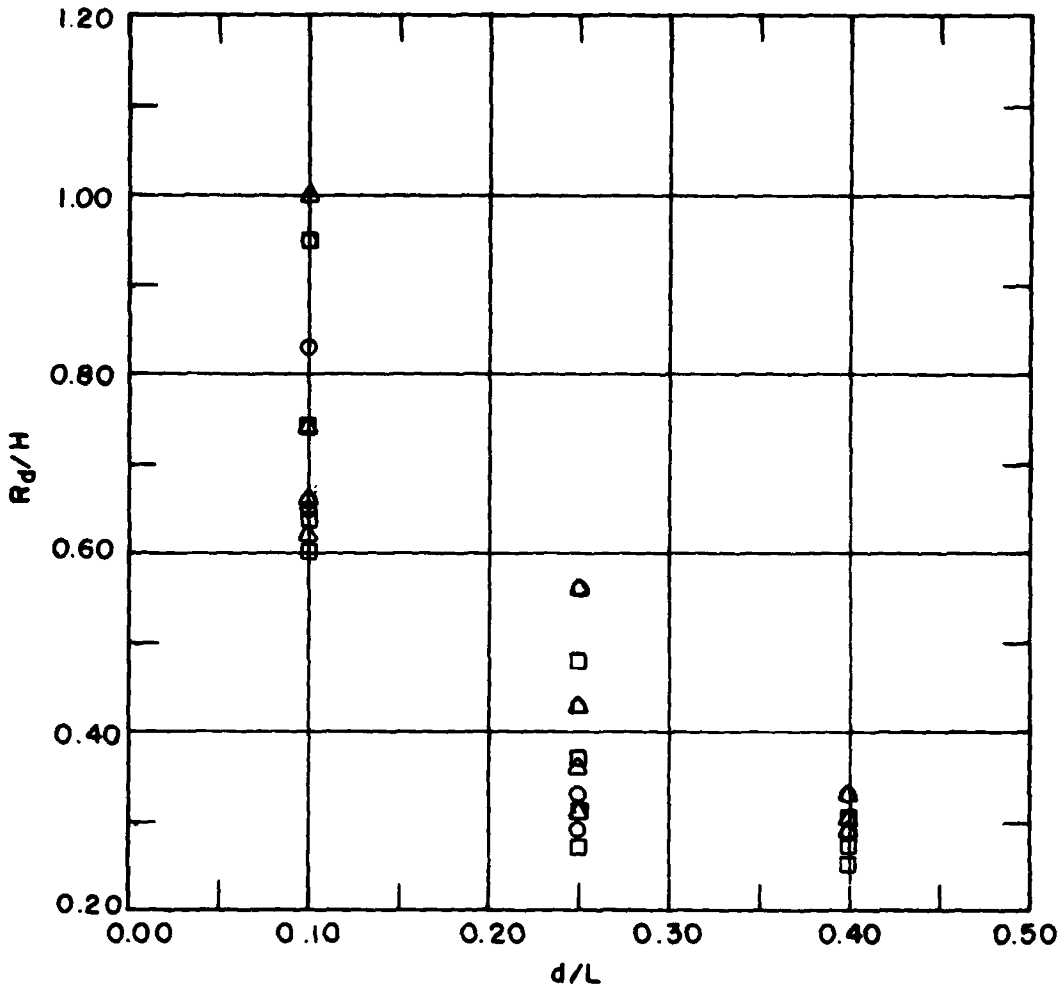


LEGEND

SYMBOL	W_l
○	$W_r/5$
△	$W_r/10$
□	$W_r/20$

COMPARATIVE VALUES OF RELATIVE RUNDOWN (R_d/H) VS RELATIVE DEPTH (d/L)

$COT\,\alpha = 1.5$
$W_l = W_r/5,\ W_r/10,$ AND $W_r/20$



LEGEND

SYMBOL	W_l
○	$W_r/5$
△	$W_r/10$
□	$W_r/20$

COMPARATIVE VALUES OF RELATIVE RUNDOWN (R_d/H) VS RELATIVE DEPTH (d/L)

COT α = 3.0
$W_l = W_r/5$, $W_r/10$, AND $W_r/20$

APPENDIX A: NOTATION

A	Surface area, ft^2
d	Water depth, ft
d/L	Relative depth
D	Damage parameter
f	Reads "function of"
F_d	Drag force, lb
g	Acceleration due to gravity, ft/sec^2
H	Wave height, ft
H/d	Relative wave height
H/L	Wave steepness
k	Coefficient
K	Stability coefficient
ℓ_a	Characteristic length of armor unit, ft
L	Length, wavelength, ft
N	Number of armor units
P	Porosity of breakwater material, percent
PT	Placement technique
R_d/H	Relative rundown
R_N	Reynolds stability number = $\left(g^{1/2}H^{1/2}\ell_a\right)/\nu$
R_u/H	Relative runup
R_u , R_d	Wave runup and rundown measured vertically above and below swl, ft
S_r	Specific gravity of an armor unit relative to water in which the breakwater is constructed
T	Wave period, sec
W	Weight, lb
W_r	Buoyant weight of armor unit, lb
α	Angle of breakwater slope, measured from horizontal, deg
$\cot \alpha$	Reciprocal of breakwater slope
γ	Specific weight, pcf
γ_r	Unit weight of an armor unit, pcf
Δ	Shape of armor unit or underlayer material
ν	Kinematic viscosity

Subscripts

a	Refers to area
d	Refers to drag
D	Refers to damage
r	Refers to armor unit
S	Refers to stability
W	Refers to water in which the structure is located
Δ	Refers to shape factor
1 and 2	Refer to underlayers

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Carver, Robert D
Effects of first underlayer weight on the stability of stone-armored, rubble-mound breakwater trunks subjected to nonbreaking waves with no overtopping; hydraulic model investigation / by Robert D. Carver. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1980.
22, [32] p., 7 leaves of plates : ill. ; 27 cm. (Technical report - U. S. Army Engineer Waterways Experiment Station ; HL-80-1)
Prepared for Office, Chief of Engineers, U. S. Army, Washington, D. C., under Work Unit 31269.
References: p. 22.

1. Armor units. 2. Breakwaters. 3. Hydraulic models. 4. Rubble-mound breakwaters. 5. Stability. 6. Water wave action. 7. Water waves. I. United States. Army. Corps of Engineers. II. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Technical report ; HL-80-1.
TA7.W34 no.HL-80-1